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Hydraulic Simulations to Evaluate and **Predict** Design and Operation of the Chashma Right Bank Canal

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EWORD

The hydraulic model application to the Chashma Right Bank Main Canal forms a part of a nine-month project entitled ***Computerizxed Planning for Operating the Chashma Right Bank Canal.*** The Pakistan Water and Power Development Authority (WAPDA) and the International Irrigation Management Institute Pakistan (IIMI-PAK) signed an agreement on June 1 1998 **to** undertake tasks that can be divided into three main components, i.e.:

1. Training of three professional staff from WAPDA and Provincial Irrigation Departments (PID), Punjab and NWFP, on the use of computers and hydraulic simulation model.

2. Simulating the of CRBC main canal design parameters: a) to develop **a** hydraulic model of the CRBC with the involvement of agency staff members, which could easily be updated after the completion of the canal, b) to evaluate the design *of* the main canal using this model for steady and unsteady flow characteristics.

3. Developing a variety of operation scenarios, including Crop-Based Irrigation Operations (CBIO), and to evaluate the hydraulic performance of alternative operation scenarios.

These **tasks** were complementary to IIMI's previous work on the **CRBC** Stage I, and an on-going activity on the Upper Swat System and the Pehur High Level Canal (NWFP). The following report mainly addresses the results of the activities carried out for Components 1 and **2.** A report, more focused on the Crop-Based Operating Schedules, ***TH€ CROP-BASED** SCHEDULES **FOR THE CRBC,*** has already been published.

For training and simulation purposes, an unsteady flow hydraulic model software, "Simulation of Irrigation Canals (SIC)", developed by a French organization, CEMAGREF, was selected. Previously, the same software was used for the CRBC Stage I and the Upper Swat System. The hydraulic part of the model, including databases for all three stages, have been provided to WAPDA, D.I Khan.

The training was successfully conducted in Lahore and D.I.Khan. At the end of the first phase, a two-day workshop was organized in November 1998, to discuss the preliminary results. Participants raised their apprehensions about the hydraulic aspects during this workshop, which are addressed in this report.

Originally, the project plan considered **only** the design situation, but later, **a** need to include the actual situation of Stages I and II in the simulation, was identified. This task **has** been accomplished with the help of **WAPDA,** D.I.Khan, and field visits by IIMI's staff. The disposition *of* the report **is** analytical, which hopefully, will contribute towards understanding the Chashma Right Bank Canal System better, and lead to better management options.

The contribution of IIMI professionals, guidance provided by Professor Gaylord **V.** Skogerboe and the efforts of staff from the line agencies are appreciated.

Dr. S. A. Prathapar
Director, IWMI Pakistan Program

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The support and cooperation of the WAPDA staff in D.I.Khan has been a major contribution to the completion of this study. The discussions with the project staff and the information provided by the Superintendent Engineers of Stages I and II during the field trips have been valuable to build the model and to understand the operational problems. Our special gratitude to three Chief Engineers of the Chashma Right Bank Project, Mr. Hafeez Ullah Jan Paracha, Mr. Muhammad Ishaq Khan and Mr. Khatid Waheed Khan for their practical support to accomplish this work.

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The authors are grateful to Prof. Gaylord V. Skogerboe, the Director of IIMI-Pakistan until December 1998, whose personal contribution and interest in the project has been a key factor to realize its completion.

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Finally, credit goes to Mr. Muhammad Iqbal Khan for the completion of this report, who has earnestly contributed in the formatting, organization and compilation of the work.

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Executive Summary

The Chashma Right Bank is a big irrigation system with a discharge capacity of 138 cubic meters per second. The main canal off-takes from the Chashma Barrage on the Indus Basin below the Tarbela Reservoir, which has a local storage capacity of 0.5 bcm. The 258 kilometer-long main canal serves 23,0675 hectares of land in two provinces, the Punjab and NWFP. An intensive distribution system has been provided an the left bank of the canal to serve the command area between the canal and the river Indus. The canal design has been planned in three phases; two operating now, while the third and largest phase is under construction. The water allowance to the command area is fixed according to the ten-daily water demand of the proposed cropping patterns, with the gross cropping intensities of 90% in winter and 60% in summer.

This report presents the steady and unsteady hydraulic behavior of the main canal, The coverage can be divided into three parts; the background of crop-based concepts recently adopted in Pakistan, description of the study area and major design features of the canal, and then, control structures (Chapters 2, 3 and 4). The description of the modules used for the analysis is presented in Chapters 5 and 7. The results of steady state analysis, sediment deposition trends, and the unsteady state behavior are presented, respectively, in Chapters 6, 7 and 8. A summary of the analytical indications is followed by the recommendations, in Chapter 9.

The first part briefs the benefits of seeking a better match between the supply and demand of irrigation water in the NWFP and the basic nature of the crop-based, or the productive-irrigation systems versus supply-based, or protective systems. The design choices developed through the process of construction from Stage I to Stage III are discussed in the context of its physical flexibility and constraints.

The steady state analysis investigates the conveyance and delivery capacity for different operational scenarios. Operational constraints on the supply side underline the implications of low and varying velocity, dead storage, over-drawing capacity of the head regulators and the non-modular and influenced operations of the control structures as the critical parameters. On the demand side, high command areas and much lower demands in *Rabi*, when compared to *Kharif*, are the major constraints. The implementation of the proposed supply and distribution range, with reference to the above-mentioned constraints, emphasize the importance of management and regulations along the main canal.

The study identifies sensitive reaches, cross regulators and distributary head regulators, which could face problems during the water stress periods. The nature of the restraint is evaluated by different simulation scenarios for about eight problematic distributary head regulators, and the scope for the improvement identified. The sensitivity analysis for the roughness-coefficient and sediment distribution patterns indicates that the free board, sufficient for the normal range of variation, could be over-stressed under the specific set of operations.

A brief analysis of the existent operations helps to understand the actual response of the canal after a few years of functioning. The simulations of Stages I and II, based on

WAPDA's survey of 1998, indicate operational constraints such as the required working heads, low sediment transport capacity of Stages I and II, as well as the available flexibility of the secondary system to function at low and variable flows. Under the current operations, the sediment deposition trends have been quite accurately predicted using a sediment module in combination with the hydraulic model. The brief sediment analysis highlights the link between operations and silt depositions in sensitive reaches of the canal.

The unsteady state simulation results confirm some of the inferences of the steady state simulations. The most important indication is that the flow variability and hydraulic instability would be the major concerns during the shift from one ten-daily operating schedule to another. Due to submerged operations of all control structures, the influence of the downstream flow conditions in a reach, or in the secondary canal, is pronounced and could prolong the transitional state of the canal. To avoid inequitable distribution and tail shortage, intensive planning and efficient operations of the system would be required. The implication of the sensitive behavior of important structures is evaluated under different situations: transitions, filling up, storage release and unplanned interventions. The response and lag times are being computed for the typical scenarios.

The main results of the analysis are summarized in the last chapter and provide guidelines to develop and implement appropriate operating procedures. Based on the major findings, a smaller set of recommendations is also compiled.

Chapter

1

1. INTRODUCTION

The Chashma Right Bank Canal (CRBC) is the first irrigation system in Pakistan, initially undertaken **as** a crop-based supply system, and then followed by the design of its physical infrastructure. **A** crop calendar in line with the existent patterns of the area was assumed with a relatively small increase of the high delta crops to ascertain allocation decisions. Due to climatic conditions of the region, the daily potential evapotranspiration vary from **3** mm/day to 8 mm/day. Therefore, the computed water demands of the proposed crops vary between 30% to 100% from the lowest to the highest demand periods although 60% of cropping intensities are proposed for *Kharif* when compared to *90%* in *Rabi.* The critical influence of the crop-based planning is 10-daily variable water allocations for the system. About 40% of the Chashma Right Bank Canal network (Stages I, II and one reach of Stage III) is currently operational, while the remainder is under design and construction.

The practical or suitable meanings of the crop-based irrigation operation have come under perpetual discussion among concerned professionals since 1987 (CRBC Stage I starts operating), and clear conclusions have yet to be reached. Nevertheless, one factor about the crop-based mode of supply; much more water is required in the network from one period to another. This leads to the variable water allocations; the range of variation depending upon the weather conditions and cropping patterns. The variable consumptive demand and allocation presents **a** real challenge for the conveyance and distribution network. The operational schedules have to be planned to maintain the gravity flow during the low allocation periods in the main and secondary system. The regutation potential of the system needs to be considerably enhanced. The live storage in the primary network becomes obligatory to feed the secondary systems, **but** must be managed to avoid the deterioration of the system through sediment deposition and operational stresses.

As the CRBC was the first system, all aspects/issues were not considered or visualized properly in the beginning. The 10-daily water allocations vary widely from 30% to 100%, while the design of the system, carried out in phases, does not follow **a** proper and uniform design criterion. The proposed design of Stage-III has responded to some of the constraints faced in Stages I and II, by selecting different design assumptions. For example, **the** roughness coefficient is taken as .018 for Stage III when compared to .016 for Stages I and II. Bigger distributary head regulators and more cross regulators are provided in Stage III, and lift irrigation has been allowed in the upper reaches of the secondary system.

The most important aspect of the operational experience of the CRBC is a request from the WAPDA and the design team of **Stage III** *to* raise the minimum inflow from 30% to 43%, and implement a canal closure period of one month for the maintenance. Whereas, the Irrigation Departments of the Province of the Punjab and NWFP are requesting a further raise in the lower limit to match the operating criterion of the supply-based systems.

To summarize, the system is currently facing three types of problems. Firstly, to apportion and regulate the discharge to keep a good match between crop-demands and supplies, while attending simultaneously to the supply side and operational constraints properly. Secondly, to complete the design of the main canal, especially of Stage III, with a proper Infrastructure, which could cope with the generic constraints of the system without adding heavy management requirements. Thirdly, to envisage and define the operational procedures, which can be implemented in the field easily without incurring heavy operation and maintenance (O&M) costs.

The approach of the current study is to present a gross, but composite overview of the design/existent physical realities, while identifying the potential and constraints of the system from the predicted responses. This is done by simulating the hydraulic behavior of the canal and its structures under different scenarios and situations. The results contribute toward answers of relevant apprehensions raised by concerned professionals. The report adapts a comparative format to provide the maximum options to managers.

The importance of the operational choices is demonstrated by the model, which in fact endorses existent field realities and the need for a more *permanent* approach to the operations of Stages I and II, even before considering the completion of Stage III.

The present analysis is limited to the primary system and offtaking structures, with a brief reference to the secondary canals. The distributary channels and the tertiary offtakes are the important components of the system, and apparently, facing less problems at the moment. A proper analysis of their current functioning could be helpful in the future operations of the whole system.

In the context of future application, it is hoped that the current study and its documentation will help in the selection of appropriate operations for the main canal. The basic set of data, information and tools developed for the design parameters would be updated after the completion of the CRBC Stage III, and then used for the actual operations.

Chapter

2

2. BACKGROUND OF THE CROP-BASED WATER DELIVERY POLICY OF CRBC

2.1 CONCEPT OF PRODUCTIVE VERSUS PROTECTIVE IRRIGATION

In comparative terms, the irrigation system with ample water allocations, intensive irrigation, and high production per unit of land are classified as productive, while the systems with thinly-spread water allocations, extensive irrigation and low cropping intensities are ranked as protective. Another protocol of supply-based versus demand-based emphasizes the mode of control availability and the manageability of the irrigation water. Usually, these classifications are used interchangeably for the big irrigation schemes.

The run of the river systems, designed to deliver continuously a fraction of river hydrograph over a specified supply period, are called supply-based systems. In most cases, like for the Indus Basin, river hydrographs vary from dry to wet ranges during the year due *to* snow-melt and rains. To maintain a constant supply to irrigation canals, the water allocation and physical capacity of diversions and canals are based on dry-period constraints; hence, the minimum requirement is taken as a capacity factor. The systems are supposed to operate within 20% of the full capacity or closed, no storage facility is provided and the minimum management interventions are required. The low cropping intensities and low delta crops are recommended. The final decision regarding land use and crops are left to the farmers.

On the contrary, demand-based systems have a minimum supply constraint, storage is usually provided, systems are designed to respond to the maximum requirement *of* the command area for the optimum cropping intensities. To satisfy a variable water demand, a flexible and management-intensive physical network is provided.

The terms “protective” and “productive” irrigation partly come from history and are conventionally used to characterize the output constraint of supply-based systems of India and Pakistan. The development *of* weir-control irrigation schemes in India started from relatively more populated areas commanded by inundation canals. The old schemes were improved or redesigned to increase flows. and irrigated areas, but still maintaining the existing cropping intensities and patterns *as* a reference. The availability of flows from the rivers was not a constraint, but the prior experience to build huge systems did not exist. In most cases many remodeling efforts had to be performed before reaching to a final design. Another consideration was to supply fresh and cheap water to the maximum population, hence, water was thinly spread in canal commands. With the process of development, experience in the field of irrigated agriculture was gained, design improved, and bigger irrigation systems were built. With time, the command area water allocation increased, but the policy to feed the maximum command area and scattered population, to build simple infrastructure and to provide the minimum management options continued.

The supply-based schemes, normally, are not non-productive for the water inputs and their production per unit of canal water is relatively high. These schemes ate not necessarily water scarce in the Indus Basin for the proposed irrigation because, for many canals, water is allocated for high delta crops and high cropping intensities in ***kharif*** (6 ***months of summer from mid-April to mid-October),*** when river supplies are at the maximum.

A comparison of the protective and productive irrigation systems as given by Jurrien (1996), is presented in Table 1. The characteristics of the systems given in the table are based on a "classical concept". The irrigation systems, in real life, have a compromise of these characteristics.

Table 1: Differences between productive and protective irrigation.

	Protective	Productive
Hydraulic		
Water duty	Low	High
Canal supplies	Constant flow	Varying flow
Control	Supply-oriented	Demand-oriented
Agriculture		
Intensity	Low	High
Seasons	One-to-one	Two-to-three
Crops	Low water demanding	High water demanding
Management		
Objectives	Poverty alleviation	Agriculture growth
Benefits	Spread	Concentrated
Optimization of	Unit of water	Unit of land
Labour	Family labor	Hired labor
Orientation	Subsistence	Market

The lacking aspects of this type of comparison are relative perspectives; localized nature of the socio-economic needs and cost-effectiveness of the management arrangements.

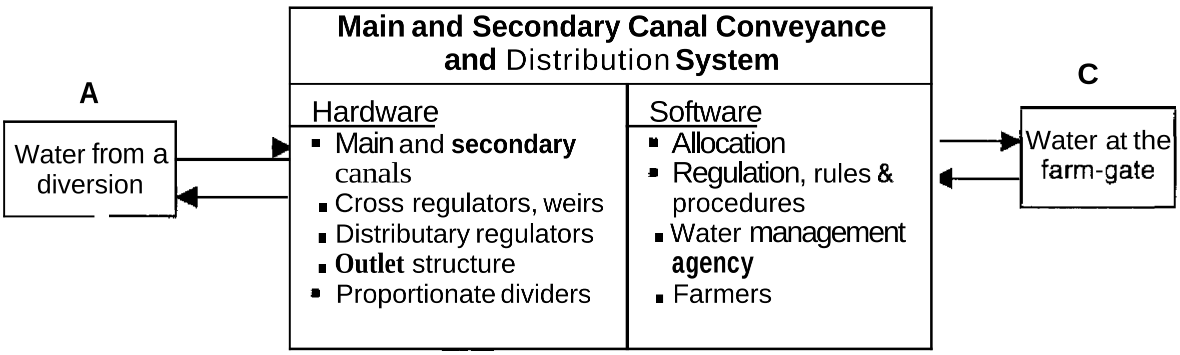
2.1.1 Water Delivery Policy and Irrigation Network

A simple input-output diagram of an irrigation conveyance system is shown below in Figure 1 (Rao, P.S. and A. Sundar, 1985; Managing Main System Water Distribution). The water released from a diversion (reservoir, barrage or main canal) to an irrigation system is processed through a combination of physical and non-physical factors, which are labeled as hardware and software, before it reaches a field. The water delivered to Box C is influenced by the original quantity in Box A and by the inter-linked influences of the components in Box B.

These links and the hardware and software components shown in **Box B** exist in any river supply network. The water delivery policy and availability of resources decides the commanding role of the supply or demand, and determines the required flexibility of the hardware and interaction between the hardware and the software components. For qualitative differentiation, an irrigation system can either be supply-driven or demand-driven, depending upon the governing nature of supply or demand. In reality, most of the irrigation systems have limitations in either way due to infrastructure (design and management) and water availability constraints. To describe these composite systems and intermediate situations, concepts/terms like "*arranged or modified demand*", "***crop-based,***" "*rotational* ***systems,***" "*intermittent flow*" and "*continuous systems*" are used.

Hence, only in extreme situations, irrigation systems are "supply-based or "demand-based", while in all other situations they are either "modified demand-" or "modified supply-based". **A** brief description **of** these terms is given below.

Figure 1: Water acquisition and conveyance in a canal system.



2.1.1.7 Demand-based Systems

This type of system should satisfy the demand of individual users, hence, the available resource must **be** more than the maximum gross demand. The designed hardware of these systems determine the supply limits. The systems are least management-intensive (no seasonal planning or in-season scheduling). The price mechanism **is** used to constrain the demand. With reference *to* Figure 1, the control on flows is from the right to the left. The requirement at "C" determines the quantity and the time of the releases from "**A**" within the constraints of "B".

2.1.1.2 Supply-based Systems

The available supply hydro-graph is the key parameter in these systems, which are mostly run of the river and non-regulated. The supply is mostly less than the demand, robust and clear operational procedures for the proportionate and equitable distribution of the available supply are followed. The systems are management-intensive at the higher levels and less responsive to interventions at the lower levels. The minimum flexibility **is** designed in the hardware and software of the system, and flows are controlled from the left to right. The water is released from "A" according to a seasonal plan having the fixed target values over the years, and is influenced by the availability in the river networks. The influence of the components in " B is constant in its nature.

2.1.1.3 Modified Demand-based Systems

The terms like *modified volumes, arranged discharge or arranged timing* are used for the systems targeted to satisfy a major part of the demand by the maximum utilization of the system's flexibility. The demands could not be met 100%, hence, are modified or curtailed by the managing agency at times. The supply constraint is not very high and the demand curve is not controlled directly. The network and management of these systems needs to be efficient, responsive and flexible. Systems are control-intensive and require a reasonably good level of management. The interaction between the boxes is two-dimensional, and the process is initiated by the box, "C".

2.1.1.4 Modified Supply-based Systems

The supply side of these systems is reinforced by reservoirs or other sources, and the management options are adopted to reduce the gap between demand and supply. Water allowances are higher in these systems. The capacity constraint is similar to that of the supply-driven system, but the flow is supplemented during peak demand periods. The systems are provided better control when compared to supply-based systems, and rigorous planning procedures are adopted for water resources management. The operations of these systems are management-intensive with a reasonable level of control required. With reference to Figure 1, the process is controlled from the left, but two-way interactions between the boxes determine the releases from "A"

By their nature, the crop-based systems defined in Pakistan are modified to supply-based rather than demand-based. For these systems, a supply range over different time periods of the year has been fixed for a standard cropping pattern. Allocations are kept higher and variable to minimize the gap between the supply and demand. Physical constraints play a critical role in the operation and management. Finally, the actual allocations are supposed to be controlled more from the supply side than from the demand side.

2.2 INTRODUCTION OF THE CONCEPT IN PAKISTAN

The concept of crop-based or productive irrigation versus the supply-based or protective irrigation was introduced in Pakistan in the late eighties during the green revolution period. In 1988, "The National Commission on Agriculture" recommended the development of plans for the distribution of irrigation supplies in accordance with crop water requirements (GOP/MoFAC 1988). The commission also suggested conducting a pilot study before taking a policy-level decision. The so-called low performance of the sector was attributed to the low water allowances, rigidity of the physical infrastructure and operation of the supply-based schemes of the Indus Basin. To provide adequate quantities of water at the appropriate stages of the crop growth, more flexible supply systems, i.e. crop-based irrigation, were suggested.

2.3 CROP-BASED SYSTEMS IN NWFP

The Irrigation Department of the NWFP decided to adopt the concept of crop-based water allocations for its three main systems: Lower Swat, CRBC and Upper Swat. The decision was facilitated by the availability of the NWFP's non-utilized share of the Indus River waters.

However, in **1990,** the Water Sector Investment Plan **(WAPDA,** 1990) questioned the utility of shifting to crop-based irrigation, and pointed out that the general improvements of the current operations of the irrigation schemes are more relevant.

The two **remodeled** systems, the Lower Swat and Stage I of the CRBC are operating since **1987.** The debate on the success or failure of these systems continues, while a few research studies carried out on parts of the system have highlighted some of their characteristics.

The characteristics of these systems, as reported by different studies, can be summarized as:

- The concept of crop-based irrigation is not clear in its contents and functioning. Two factors aiding this confusion are:

    1. the actual cropping patterns, the crop demand hydrographs, and the actual supplies differ from the proposed patterns and allocations originally envisaged for the command area; and

    2. the major parts of the systems are designed for the full supply operations and top-down management, like any other system in Pakistan. The flexibility provided in terms of bigger structures and more control in the main canals has not facilitated the operations.

- The allocations are made for higher cropping intensities, i.e., 150% (CRBC) to 170 percent, and for the variable supplies over the year, in the range of 3 cfs to 8 cfs per 1,000 acres.

- The actual cropping patterns develop with a higher percent of high delta crops (rice about 30%, vs. 2% proposed) in the first PC-1. The canal supplies are excessive most of the time [high Relative Water Supply **(RWS);** Mr. Helsema computed RWS to be more than i 2 for Sheikh Yousaf Minor, WAMA 1997; RWS in sample outlets of CRBC vary between 1 to **1.8** (IIMI Final Report, 1994)].

- The main canals have been designed for the peak water demand with the provision of cross-regulators and gated distributary head regulators. The secondary and tertiary systems are designed without additional control structures, but mostly with substantially higher capacities than the maximum authorization. In both systems, the excessive supplies head-tail inequity and a need to enhance managerial control, have been identified **(WAMA 1997; IIMI 1994).**

- In these systems, the accumulated percolation and seepage losses are much higher than the Indus Basin average. This factor is less problematic in the LSC, but may *be* very serious in the CRBC due to **a** difference **in** topographies and natural drainage characteristics **(WAMA** 1997).

- The average yield of the major crops has increased substantially and is higher than the national average in both systems (IIMI 1994, WAMA 1997).
- The old O&M procedures (hierarchical top-bottom control, *warabandi*) are not established in these systems. The new operational procedures have not been envisaged yet, leading to the non-existence of a proper monitoring and evaluation set-up. The canal structures are not calibrated, the daily gauge registers are not properly kept and the accuracy of the record keeping is poor.
- In practice, a new type of control with a higher involvement of the users and the local gate operators is emerging in the systems that are currently at the intervention level, and is inconsistent with the management procedures.

2.3.1 Crop-Based Vision for CRBC

2.3.1.1 PC-1 and Water Apportionment Allocations

These two documents indicate that the CRBC irrigation system has been planned to allocate and utilize river supplies according to the demand of the planned cropping patterns in the area.

The PC-1 mentions two factors indicating the crop-based nature of the system:

i. Daily crop water requirements have been computed for proposed cropping patterns accumulated to 60 percent intensities in *Kharif* and 90 percent in *Rabi*. The US Weather Bureau Method was used for these computations (Page 23 PC-1 1981). PC-I gives the crop calendar and 10-daily requirements for each crop. The uniform cropping patterns are considered for the whole command area.

ii. It is mentioned that the main canal will be provided with cross-regulators, each supporting a group of 3 to 4 distributaries.

The Water Apportionment Accord of March 1991 authorized 10-daily allocations for the CRBC based on evapotranspiration estimates. The accord has fixed the share of the Punjab Province and NWFP in the light of PC-I (1981)[1] and the Feasibility Study of 1990.

2.3.1.2 Crop Water Requirements of CRBC Command Area

The crop water requirements were first estimated by the Planning Commission in 1981 (PC-1). Major changes were not suggested by the Technical Assistance Missions (TAM) in 1982 and 1987. The 10-daily water allocation by the Water Apportionment Accord (WAA) closely matches the PC-1 requirements. However, according to the design consultants of Stage III, two other reports, Feasibility Report (1990) and Staff Appraisal Report compute different 10-daily requirement patterns. The different sets of 10-daily crop water requirements are shown In Table 2. The PC-I accesses the crop water requirements using WAPDA's computer

[1] The same data are repeated in the later versions of the PC-I, 1991, 1996

program Delta-2, which was used in 1978 to access the crop water requirements for the Indus Basin Model Revised (IBMR).

The net seasonal volumes allocated by the WAA are the same as given by PC-I (Figure 2). Similar cropping patterns have been adopted for both provinces, while a slight shift in the supply hydrograph has been computed due to climatologic variations (rainfall and humidity levels) between D.I.Khan and D.G.Khan. According to the WAA, there is no other canal closer to the CRBC.

Figure 2: 10-daily Allocations as per PC-1 and Water Apportionment Accord.

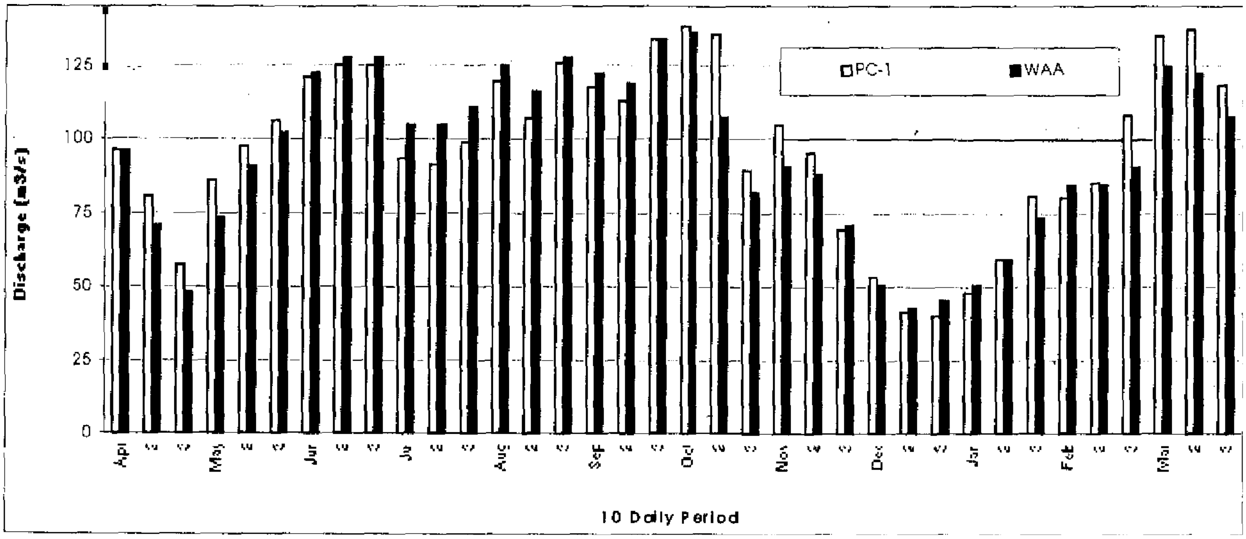


Table 2: Water requirements as per PC-1, SAR, TAM and Water Apportionment Accord.

Months and 10-day Period		s for 570.000 acres CCA					
		PC-1 1981 (m^3/s)	TAM 1990 (m^3/s)	ADB Staff (SAR) Appraisal Report (m^3/s)	Water allocation as per WAA NWFP (m^3/s)	Punjab (m^3/s)	Total (m^3/s)
April		96	101	92	59	37	96
	2	81	65	59	48	23	71
	3	58	43	39	34	14	48
May		86	55	51	54	20	74
	2	97	85	81	59	31	91
	3	106	105	100	65	37	102
June		120	135	129	74	48	122
	2	125	141	133	76	51	127
	3	124	164	152	76	51	127
July		93	137	117	57	48	105
	2	91	137	115	57	48	105
	3	99	176	152	59	51	110
August		119	185	172	74	51	125
	2	107	195	182	65	51	116
	3	125	201	189	76	51	127
September		117	176	171	71	51	122
	2	113	172	167	68	51	119
	3	134	155	150	82.	51	133
October		138	101	104	85	51	136
	2	135	69	68	82	25	108
	3	90	77	77	54	28	82
November		104	67	65	65	25	91
	2	96	76	74	59	28	88
	3	69	78	75	42	28	71
December		53	44	42	34	17	51
	2	41	50	47	25	17	42
	3	40	52	52	25	20	45
January		48	64	60	28	23	51
	2	59	64	60	37	23	59
	3	81	69	65	48	25	74
February		80	103	96	48	37	85
	2	85	88	81	54	31	85
	3	108	72	66	65	25	91
March		134	118	108	82	42	125
	2	137	104	95	85	37	122
	3	118	93	85	74	34	108
Kharif Total(BCM)		1.63	2.10	1.95	1.00	0.66	1.66
Rabi Total(BCM)		1.40	1.20	1.14	0.86	0.45	1.13
Total Annual		3.03	3.29	3.09	1.86	1.11	2.97

Chapter

3

3. CHASHMA RIGHT BANK IRRIGATION SYSTEM

3.1 STUDY AREA

The Chashma Right Bank Canal is a partially completed large irrigation system spread over two provinces of Pakistan. It is fed **by** the Indus River through the Chashma Barrage. The total command area of the canal is 230,675 hectares (570,000 acres). The canal has been ptanned to deliver discharges in the range of 138 cumecs (4879 cusecs) maximum to 41.5 (1464 cusecs) cumecs minimum, over the year. The main canal length **is** 258 km (170 miles). The distribution system is about 1,100 km (700 miles) long, and 83 secondary channels irrigate *a* narrow strip of land between the CRBC main canal and the river Indus. The command area is compact and irrigation-intensive when compared to other systems of the bash. The Project layouts prepared in 1982 and revised in 1999 are shown in Figures 3a and 3b.

The arid to semi-arid area of the CRBC command is situated between the Indus River and the Suleman **Range.** The average annual rainfall ***is*** about 256 mm (10 inches), while the average evapotranspiration from the free surface varies from 1,400 mm to 1,650 mm (55 to 65 inches). The maximum temperature **goes** to 100 F.

The soils of the area are clay to silty-clay. A part of the area with heavy soil composition, scattered sand dunes and gravel are found in the vicinity of the northern and western hills. The natural surface slope **vary** from the foot hills towards the river at the rate of one meter to a kilometer ***(5*** ft to a mile).

The ground water table depth varies in a range of 180 cm (6 feet) near the river to about 900 cm (30 feet) in the plains. The ground water quality **is** moderate to **bad** in **the** project area, with fresh water pockets.

A few streams, the largest of which is the Gomal River of about 3 cumecs (100 cusecs), emanate from the Suleman Range, while the numerous flood channels traverse the land between the mountain range and the river. Traditionally, floodwater has been the major source of irrigation water supply, managed and used locally **by** the farmers. Before the implementation of the current project, two canals with a limited water supply were available to irrigate about 50,000 ha (124,000 acres) of land.

The population of the command area is about 1,000,000 persons; a density of 348 persons per square mile.

3.2 BRIEF PROJECT HISTORY

- The project was first approved by ECNEC on November 30, 1978, at a capital cost of Rs. 1570.43 million), The cost was revised to Rs. 3477.55 million in February 1982, again to Rs. 4275.55 million in 1986 and to Rs. 10213.27 million in November 1991.
- The project approved by ECNEC in 1991 in the composite form for Stages I, II & III was for about Rs 10,213.3 million, with a 21% share of GOP. The cost was again revised to Rs.15,532.6 million, the increase being mainly towards the cost of Stage III (PC I, 1991).
- The cost of the project includes 16% interest rate, white the benefit-cost ratio is 1.28:1; computed at the interest rate of 12%. The estimate for the Internal Economic Rate of Return is 14.2%.
- The gross values of produce estimate is Rs. 7759 million per year and the net value is equal to Rs. 3821 million per year. The estimated O & M cost to be paid by the farmers is Rs 254 per acre (about 627 Rs./ha.).
- Stage I was commissioned in 1978 and Stage II from 1992 to 1994. The construction of Stage III started in 1998 and the scheduled completion period is December 2000. A 6 km-long stretch of Stage III has been constructed as a part of Stage II and is operating since 1994. This reach feeds 3 secondary canals of the total discharge 9.57cumecs (338 cusecs), while the remainder of the Stage III system is in the design and contracting phase.
- The estimated economic life of the project is 50 years (PC-I, 1998).

3.3 MANAGEMENT SET-UP OF THE SYSTEM

- Officially, three departments are responsible for the operation and maintenance of the system. The Water and Power Development Authority (WAPDA) for the main canal, Provincial Irrigation Departments of the NWFP and Punjab for the secondary canals of their respective provinces.
- WAPDA releases the water from the Chashma Right Bank Canal according to an estimated demand and conveyance pattern. All cross-regutators and escape structures along the main canal are operated by WAPDA to maintain a required water level, while distributary head regulators are operated by WAPDA in consultation with the Irrigation Department.
- The recommended Monitoring and Evaluation (M&E) set-up has not been finally established in Stages I & II, especially, for the distributary head operations, The hourly gauge readings are recorded for cross-regulators and escapes. The frequency of operations of these structures is a few times a month to a few times a day.
- A special monitoring project has been implemented on the canal from 1987 to 1995 by the Alluvial Channel Observation Project (now named: International Sediment

Research Institute Pakistan, ISRIP). The **AD8** sponsored the program just after the inception of the canal when heavy seepage occurred in the unlined section along both banks of the canal. The morphological observations (water and sediment inflow and outflow, cross-sectional survey) of Stage **I** have **been** recorded three times **a** year for a **couple** of years.

- **The WAPDA** and Irrigation Departments are responsible for the maintenance of the main and secondary canals. **An** appropriate set-up for the maintenance of the system has not been developed yet.

- At the present stage, farmers are involved in the water release to the secondary channels and the water distribution to the tertiary system **by** influencing the management at these levels. Otherwise, a formal institutional set-up for farmers' interventions or negotiation with the agencies does not exist **so** far.

Chapter

4

4. DESIGN OF THE MAIN CANAL CONVEYANCE AND DISTRIBUTION SYSTEM

The construction of the Chashma Right Bank Canal has been implemented in three stages as described in the project history. An important physical feature of the CRBC is the provision of structures to carry rainwater conveyed by numerous hill torrents from the Suleman Range across the canal.

4.1 MAIN CANAL

A layout of the main canal structures is shown in Figure 3a and 3b. The salient design characteristics of the CRBC are:

- At the main canal level, cross-regulators, escapes and gated off-takes have been constructed to achieve and maintain the required water elevations at variable supplies, while at the distributary level comparable operational flexibility has not been provided.

- Twenty-three cross-drainage structures have been provided either as super-passages or siphons for the flood runoff. Some of these channels are bigger in capacity than the CRBC main canal itself.

- A tile drainage project has been implemented in the upper part of the canal command after the commissioning of Stage I in 1987. Surface and flood carrier drains exist in the area, but due to high seasonal floods, soil physiography and saline ground water, the tile drainage network is important for the project and may need to be extended.

- The first thirty-five kilometers (RD 120,000) of the CRBC main canal are unlined. The canal is in "fill" in this section. On the average, the Water Surface Level (WSL) is about 2 meters (6 ft) higher than the Natural Surface Level (NSL). The average bed width of the canal in the unlined section is 30.48 meters (100 ft), the average depth 2.44 meters (8 ft), and the bed slope is one in 7,000.

- The first control-structure of Stage I is located about 30 km downstream of the head regulator. This combined structure consists of an escape, a cross-regulator and a sift excluder. The second control structure of the canal is at the end of Stage I, which is a couplet comprising of an escape and a cross-regulator on a siphon. The siphon passes underneath a drain with a capacity of 850 m^3/sec.

- The unlined section of Stage I is in "fill", the three distributaries have a twin set of upper and lower head regulators, the latter being installed after the trial run of Stage I when water elevations in the canal was found lower than the sills of the original structures. It is assumed that lower outlets are temporary and the **canal would** achieve **the** design section within the next few years.

- The lined section of Stage I, from 36 km (RD 120,000) to **78 km (RD** 257,000) **has** been designed using Manning's formula **and a** roughness coefficient of .016. **This** section is **in** cut, the full supply water level (FSL) being at the level of **NSL. The** average bed-width of this **section** is 13.72m **(45** *ft),* the **average depth** 4.87m (16 **ft)** and the **bed slope** one in 14,000. The six distributaries **off-take** from the lined section and **one** cross-regulator **has** been provided to regulate the flow **in** the **low supply** period.

- Stage II of the canal is 36 kilometers long, from 78 km (RD 257,000) to **115** km (RD 378,000). The assumed roughness in this section is **,016,** the bed width-depth ratio 2.3 and the average velocity in the range of 1.1 m/sec (3.6 ft/sec). **Three** cross-regulators have been provided to feed nineteen distributaries. The command area of this section **is** high, all head **and** cross-regulators operate in submerged conditions and storage **is** required in each reach to **feed** the secondary canals during low flow periods.

- The main and **secondary** canals and the secondary and tertiary head regulators of Stages I & II have been designed to operate at the full **supply** discharge, which **is the** maximum authorization according to the **WAA.**

- **Stage III of** the CRBC is 144 kilometers long, starting from RD 378,000 **(ft) to** RD **848,700 (ft).** The design approach for Stage III has been modified to provide more regulation at the main canal level. Three major changes are:

  i. **The** roughness is increased from .016 to .018.

  ii. The main canal regulation *is* designed to deliver the full **supply** to the secondary canals at 43% of the design discharge **and** ***67%*** of the maximum **levels** in the main canal **by** operating the cross-regulators.

  iii. Sixteen cross-regulators have been provided in Stage III to feed the secondary canals through the backwater curve.

Table 3: A stage-and-province-wise summary of the design parameters.

Design parameters	Stage I	Stage II	Stage III	NWFP	Punjab
CCA (ha)	60728	38041	131930	141643	89032
Canal length (km)	78	36	144	156	102
Maximum allocation (m^3/Sec)	138	101	79	87	52
Minimum allocation (m^3/Sec)	42	31	24	27	16
Maximum delivery to CCA (m^3/Sec)	32	22	77	81	50
Total seepage (m^3/Sec)	5.1	0.54	1.9	6.1	1.42
Assumed roughness	.016	.016	.018		
Bed width/depth ratio	2.62	2.31- 2.1	2.06-.67		
Maximum velocity (m/Sec)	1.22	1.10	0.94-0.70		
No. of x-regulators	2	3	16	8	13
No. of distributaries	12	16	54	45	37
Head regulator design criteria	100% FSL	100% FSL	67% FSL		

4.2 DIVERSION FACTOR FOR THE SYSTEM

A diversion factor computed by PC-1 (1982) is based on the following assumptions for the canal losses:

Farm losses	=	25%
Watercourse **losses**	-	10% (with 70% lining)
Main canal, distributary and minor	-	12%
Diversion Factor	-	100/(.75*.9*.88) = 168%

This gives an overall water transfer efficiency of 59.4%. However, different studies have estimated different values for the expected losses. The estimates of the Water Transfer Efficiency are quoted in IIMI's report, *APPLICATION OF CROP-BASED IRRIGATION OPERATIONS TO* ***CHASHMA RIGHT BANK CANAL*** *(October* ***1998,*** *page* **72).**

Table 4. Water Transfer Efficiency and seepage loss estimates for CRBIP.

Losses (cubic meters) and Efficiency Components	WAPDA PC-I(1981)	WAPDA (Oct. 1982)	ADB 1982	ADB[a] 1987	SPMP[b] 1988
Losses					
Main Canal	**10.00**	**10.00**	**9.91**	**9.91**	**4.67**
Distys/Minors	**6.09**	**7.31**	**9.51**	**7.31**	**5.64**
Paharpur	*0*	**2**	**2**	**2**	**9**
Evaporation	*0*	0	1.13	0.99	0.54
Wastage	0	*0*	7.08	*0*	*0*
Total Losses	**16.08**	**19.03**	**29.90**	**20.19**	**19.65**
Efficiency Components					
Canal Efficiency	**0.886**	**0.886**	**0.789**	**0.857**	**0.861**
Watercourse Efficiency	**0.9**	**0.9**	**0.77**	**0.82**	**0.84**
Farm Application Efficiency	**0.75**	**0.75**	**0.75**	**0.75**	**0.77**
Overall Transfer Efficiency	**0.598**	**0.584**	**0.455**	**0.527**	**0.557**

a = ADB consultant revised the estimation of 1982.
b = System's Performance Monitoring Project.

Table **4** shows that the expected efficiency varies from 45% to 60% based on the assumptions for the losses. The monitoring data of the CRBC Stage **I, by** ACOP (Alluvial Channel Observation Project, presently ISRIP) and IIMI in 1992, has shown seepage losses from the lined section to **be** higher than the design values (Performance of CRBC, Zaigham Habib, Carlos Garces).

4.3 BED WIDTH, DEPTH RATIO FOR THE MAIN CANAL

To maximize hydraulic efficiency, the channel width to *the* depth ratio should result in a maximum hydraulic radius. The recommended minimum b/d ratio for the lined canals is in the range of 1 to 2 for the discharge range of the CRBC. These **values** come from the limited international experience with **good** quality lining, high velocities and no sediment deposition situations. For the CRBC, the bed-width to depth ratio varies from ***2.8*** to ***0.5*** for the lined section. Figures 4 a, b and c show the theoretical (Canal System Automation Manual), and the CRBC **b/d** for a discharge variation of 120 *to* **4** cubic meters.

During most of the discussion, engineers have commented about the hydraulic efficiency and the **b/d** *ratio.* **A** detailed analysis **of** the **b/d** ratio is outside the **scope** of **this** study, while a **need** to establish empirical **values** for b/d in the context of the Indus Basin is identified.

Figures 4(a-c): Theoretical and actual bed-width and depth parameters.

Figure 4.a: Theoretical minimum b/d ratio recommended for the good quality concrete lining

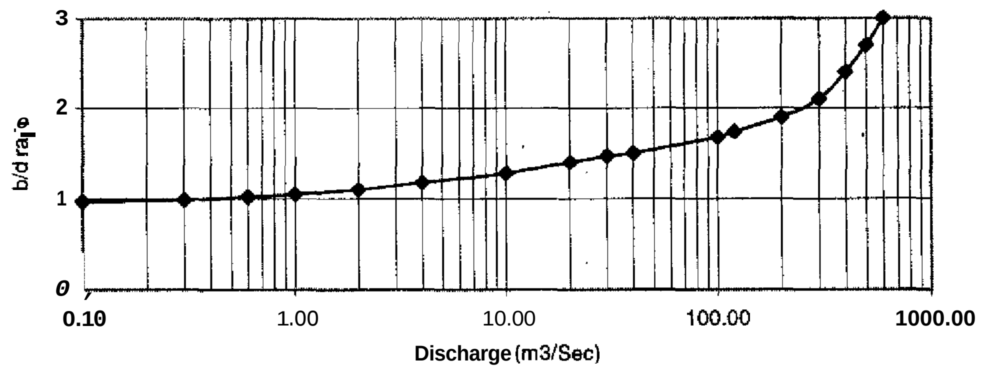


Figure 4.b: Actual b/d ratio for CRBC Lined Canal

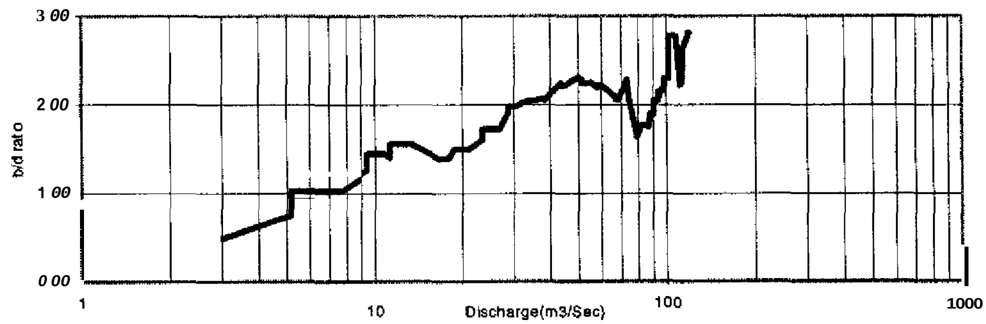
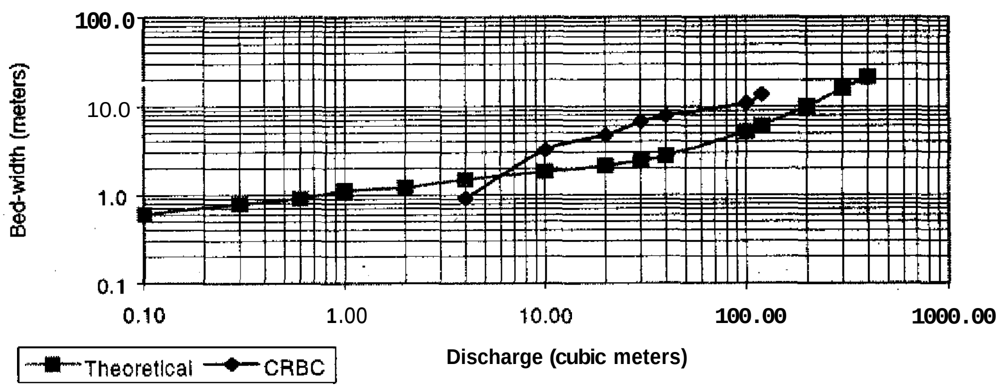


Figure 4.c: Theoretical and Actual bed-width versus discharge



4.4 DISTRIBUTARY HEAD REGULATORS

The design criteria for distributary head regulators of Stages I and II consider the full supply operations as the target. The working heads are computed with reference to the full supply water levels. The position of the cross-regulator and the level of the downstream command area are the two constraints on the working head, which compel the presence of higher working heads in the main canal. The crest levels of the head regulators are kept a couple of feet higher than the main canal bed level due to the same reason, and hence, their sediment sharing capacity is low. The bed in the unlined section, after construction, was much lower than the design bed, but, has raised substantially during the past twelve years. For four distributaries, off-taking from the unlined section, the temporary lower pipe off-takes where introduced in 1987, because their standard head regulators were not able to draw water.

The proposed design of control and diversion structures of Stage III intend to deliver full supply to the secondary system at 67% of the full supply water levels and 43% discharge in the main canal. The command areas are again quite high and lift irrigation is provided in the first reaches of the distributaries. The design dimensions of the regulators are given in Table 5. A few hydraulic characteristics of distributary head regulators are:

- The head regulators are not designed strictly as the proportionate flow dividers, and the share of the discharge delivered will be controlled by the gate operations.
- Pipes are selected as off-take structures for distributaries instead of square barrels or culverts. The pipe structures are very sensitive at lower and higher flows and the role of gates to handle the flow variability will be quite pronounced.
- The constraint of command area, natural surface slope and design assumptions has resulted into high crest levels and the non-modular functioning of the head regulators.
- The constraints imposed by the working head, command area level and the variability of flows are addressed by increased flexibility provided by the big size head regulators.

Table 5: Dimensions of cross and head regulators.

Name of Distributary	Distance from Head	No of Gates	Barrel		Pipe Dia (m)	Bed Level (m)	Crest Level (m)	Downstream WSL
			Width (m)	Height (m)				
Stage-I Regulator	29498	12	3.35	2.07		186.6	188.7	
Takarwah Link Upper Barrel	30174	1	1.22	1.22		185.3	188.6	
Takarwah Link Lower Pipes	30175	2			1.14	185.3	186.6	
Kot Hafiz Link Upper Barrels	34964	2	0.91	1.22		185.5	188.1	
Kot Hafiz Link Lower Pipes	34966	2			1.22	185.5	186.4	
Stage-I Trans Weir	36575					185.3	187.5	
Kathgarh Link	39898	1	0.91	1.13		184.7	185.7	
Saiduwali Minor	44440				0.38	184.4		
Link Feeder Upper Barrel	51762	2	1.22	1.22		183.9	187.0	
Link Feeder Lower Barrel	51763	1	1.22	1.22		183.9	185.3	
Disty-1	61325	1			0.69	183.2	186.4	
Disty-2	68295	1	0.91	0.91		182.7	185.5	
Disty-3	72335	2	0.91	1.22		182.4	185.1	
Disty-4	76253	2	1.37	1.22		182.1	184.8	
Stage-I Tail Regulator	78344	5	3.00	4.82		182.0	182.6	
Disty-5	**80739**	**2**	**1.37**	**1.22**		**181.6**	**184.6**	**186.2**
Disty-5A	83416					181.4		
Disty-6	**85123**	**2**	**1.37**	**1.22**		**181.3**	**184.2**	**185.9**
Disty-7A	**87325**	**1**			**0.46**	**181.2**	**184.5**	**185.2**
Stage-II X-Regulator1	87362	2	7.70	4.57		181.2	182.1	
Disty-7	**91252**	**2**	**1.37**	**1.37**		**180.8**	**183.7**	**184.9**
Disty-7B	**92743**	**1**			0.30	**180.7**	**184.3**	**185.1**
Disty-8A	95555	1			0.76	180.5	184.0	
Disty-8B	**96378**					**180.5**		**184.9**
Disty-8	99060	1			0.61	180.3	183.9	
Disty-9	**100248**	**1**			**0.69**	**180.2**	**183.2**	**184.7**
Disty-10	**103353**	**1**			**0.91**	**180.0**	**182.8**	**184.5**
Stage-II X-Regulator2	**103518**	**2**	**7.62**	**5.26**		**180.0**	**180.9**	
Disty-10A	**104581**	**1**			**0.61**	**180.0**	**182.9**	
Disty-11	**108209**	**1**	**0.91**	**1.22**		**179.6**	**181.6**	**184.0**
Disty-11A	**108759**	**2**	**1.37**	**0.91**		**179.4**	**182.3**	
Disty-12	**111008**	**1**	**1.83**	**1.22**		**179.3**	**182.3**	**183.8**
Disty-13	**112479**	**2**	**1.37**	**1.22**		**179.2**	**182.2**	**183.8**
Stage-III X-Regulator3	**115214**	**3**	**5.28**	**4.37**		**179.1**	**180.0**	**183.6**
Disty-14	**115976**	**2**	**1.22**	**1.22**		**178.9**	**181.3**	**183.6**
Disty-15	**118068**	**2**			**1.37**	**179.0**	**181.1**	
Disty-16	**121118**	**2**			**1.22**	**178.7**	**181.0**	**182.5**
Disty-16A	**123784**	**1**			**0.91**	**178.5**	**181.2**	**182.5**
Disty-17	**129392**	**1**			**1.37**	**178.0**	**180.1**	**182.0**
Stage-III X-Regulator1	**129410**	**2**	**6.40**	**4.86**		**178.0**	**178.2**	
Disty-18	**136150**	**1**			**1.68**	**177.4**	**179.2**	**180.8**
Disty-18A	**138158**	**1**			**1.68**	**177.4**	**179.0**	**180.6**
Disty-19	**141409**	**1**			**1.52**	**177.0**	**179.0**	**180.4**
Disty-20	**144524**	**1**			**1.37**	**177.0**	**178.9**	**180.1**
Stage-III X-Regulator2	**144545**	**2**	**6.40**	**4.60**		**177.0**	**177.2**	
Disty-20-A	**148198**	**1**			**1.37**	**176.7**	**178.6**	**179.9**
Disty-21	**150686**	**1**			**1.37**	**176.6**	**178.4**	**179.7**
Disty-22	**153736**	**1**			**1.52**	**176.4**	**177.9**	**179.4**
Disty-23	**156574**	**1**			**1.37**	**176.2**	**177.9**	**179.2**
Stage-III X-Regulator3	**156596**	**2**	**5.49**	**4.47**		**176.2**	**176.4**	

Name of Distributary	Distance from Head	No of Gates	Barrel		Pipe Dia (m)	Bed Level (m)	Crest Level (m)	Downstream WSL
			Width (m)	Height (m)				
Disty-24	161444	1			0.91	175.7	177.8	178.7
Disty-24A	162347	1			1.07	175.6	177.6	178.6
Disty-25	163810	1			1.68	175.6	176.8	178.4
Stage-III X-Regulator4	165052	2	5.49	4.33		175.5	175.7	
Disty-26	167685	1			0.91	175.3	177.3	178.2
Disty-27	170157	1			1.52	175.1	176.4	177.9
Disty-28	174702	1			1.52	174.7	176.1	177.5
Stage-III X-Regulator5	174721	2	4.88	4.09		174.7	175.0	
Disty-29	177640	1			1.68	174.3	175.5	177.1
Disty-30	181633	1			1.68	174.1	175.1	176.7
Stage-III X-Regulator6	183632	2	4.88	3.94		174.0	174.4	
Disty-31	186492	1			0.91	173.7	175.6	176.4
Disty-32	190140	1			1.37	173.4	174.8	176.1
Disty-33	192519	1			1.37	173.3	174.6	175.9
Stage-III X-Regulator7	192538	2	4.88	3.87		173.3	173.6	
Disty-34	196236	1			1.37	172.9	174.2	175.5
Disty-35	198736	1			1.52	173.1	173.9	175.3
Disty-36	202744	1			1.52	172.6	173.6	175.1
Stage-III X-Regulator8	202762	2	3.96	3.69		172.6	172.9	
Disty-37	207378	2			1.52	172.2	173.2	174.7
Disty-38	213146	1			1.37	171.9	172.9	174.2
Stage-III X-Regulator9	213165	2	3.96	3.57		171.9	172.1	
Disty-39	217101	1			1.68	171.6	172.3	173.9
Disty-40	219799	1			0.91	171.4	173.0	173.8
Disty-41	222666	1			1.52	171.2	172.1	173.5
Stage-III X-Regulator10	222791	2	3.96	3.35		171.3	171.3	
Disty-42	226000	1			1.07	171.0	172.2	173.2
Disty-43	231654	1			1.37	170.5	171.5	172.8
Disty-44	232898	1			0.91	170.5	171.9	172.7
Stage-III X-Regulator11	234391	2	3.05	3.05		170.4	170.8	
Disty-45	236916	1			1.68	170.2	171.0	171.9
Disty-46	239776	1			0.91	169.9	170.8	171.7
Stage-III X-Regulator12	240223	2	3.05	2.20		169.9	170.1	
Disty-47	241150	1			1.68	169.6	170.2	171.2
Stage-III X-Regulator13	241168	2	2.44	2.18		169.6	170.0	
Disty-48	244899	1			0.91	169.1	169.8	170.7
Disty-49	248107	1			1.37	168.7	169.3	170.2
Stage-IIIX-Regulator14	248126	2	2.44	2.05		168.7	168.9	
Disty-50	250812	1			1.37	168.2	168.7	169.7
Disty-51	252383	1			1.68	168.0	168.3	169.3
Stage-III X-Regulator15	252401	1	3.05	2.07		168.0	168.3	
Disty-52	256100	1			1.68	167.5	167.7	168.7
Stage-III X-Regulator16	258684	1	2.44	1.97		167.2	167.8	
Disty-53	258985	2			1.37	167.2	168.6	168.7

Chapter

5

5. THE HYDRAULIC MODEL "SIMULATION OF THE IRRIGATION CANALS (SIC)"

The SIC software, Simulation of Irrigation Canals, is a mathematical hydraulic simulation model developed by a French Institute, CEMAGREF, Montpellier. The model could provide answers to major problems that confront the canal manager, by:

- simulating the steady and unsteady state hydraulic and operational conditions in an irrigation canal,
- comparing and testing physical modifications in the canal topography or control structures, and
- evaluating new management rules, once they are defined and formulated.

A user-friendly interface is provided at each level to process input data and output information. The SIC has been organized around three units, which, respectively, handle *canal topography, steady state conditions and unsteady flows.* This chapter briefly presents the modeling process and the basic hydraulic laws used in three units of the SIC model.

5.1 UNIT 1 – TOPOGRAPHY MODULE

A main canal network conveys and distributes water from a source, (reservoir or river diversion) to various off-takes, (secondary and tertiary canats). These networks are mostly branched at three levels. The main canal conveying water, secondary canals distributing water to the tertiary system and small tertiary channels scattered all along the distributary canals, taking water to the command *areas.* The essential physical components of the system are control works, regulators, distributors, gated and un-gated diversions. The hydraulic modeling of the whole, or part of the network takes into account the real canal topography, the canal network topology and its geometric description. Unit 1 manages all the topographic components used by the model.

5.1.1 Description of the Hydraulic Network

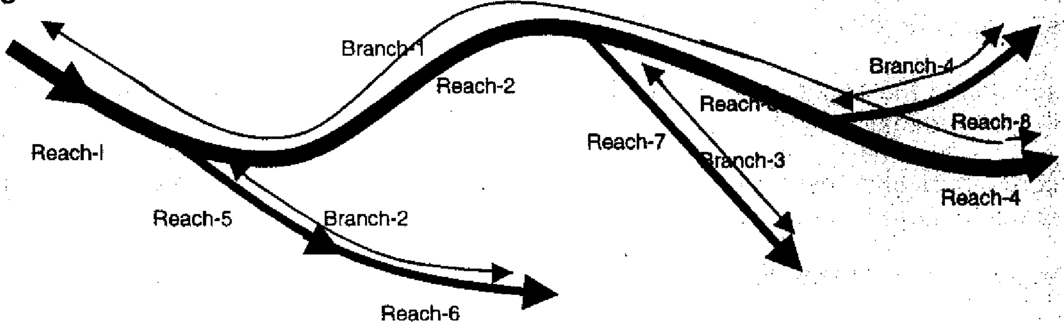
The canal network in hydraulic models is defined with reference to two main components of the canals, off-takes and x-regulators. A canal reach is either between two off-takes or between two cross-regulators.

In the SIC Model, the location of each off-take is defined *as* a nodal point. The hydraulic network is divided into homogeneous reaches located between an upstream and a downstream node. The user may divide any part of the canal into several reaches in order to take into account some particularity, for instance, a change in roughness or administrative unit Regulating and control devices have been integrated within a reach and do not need any special division This approach facilitates the modeling of the hydraulic transition from free flow conditions to submerged conditions for these devices,

5.1.2 Choice of Branches

A branch is a group of reaches serially linked to one another. Users can group the reaches of their choices into branches. A branch in the model is not the same as a branch in a canal. Figure 5.1 illustrates how a canal network is subdivided into reaches and branches.

Figure 5.1: The selection of canal reaches and branches.



5.1.3 Upstream and Downstream Boundary Conditions

The inflow discharge is specified at the first node of the network, while a rating curve (Q-H relations) needs to be given at the last (downstream) node of the network, as the boundary conditions. The first node of the canal is defined upstream of the first structure, so it is a starting point where the water taken by the head regulator is available. The inflow hydrograph is defined at this point.

Downstream boundary conditions are very important for a long and sensitive canal with variable flow conditions. The depth-discharge relations at this location are the starting point for the flow profile and must always be sub-critical.

5.1.4 Description of the Cross-sections

The reach geometry is defined by the cross-section profiles, characterizing the shape and volume of the canal at a particular location. The elevations are indicated with reference to a unique datum (benchmark for the bed level with reference to the sea level) along the canal. The SIC model provides a facility to enter cross-sections in three different formats:

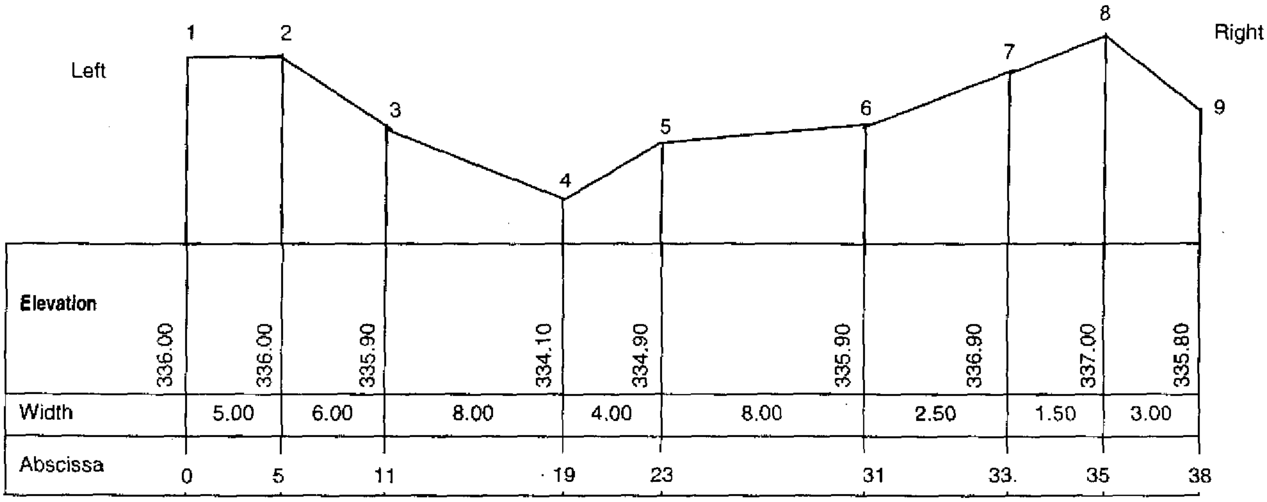
- abscissa-elevations;
- width-elevations; and
- parametric forms.

The data entry format can vary from one section to another within a given reach.

5.1.5 Abscissa-elevation

Cross-sections provided by the surveyor are usually in the abscissa-elevation format, **as** shown in Figure 5.2, The section may **be** introduced either from the left bank or from the right bank. The model **does** not process the bank itself, and **all** points after the highest bank **level are** ignored; Points **1** and 9 in Figure 5.2. Hence, the entry of the last point is important to have **a** correct cross-sectional area.

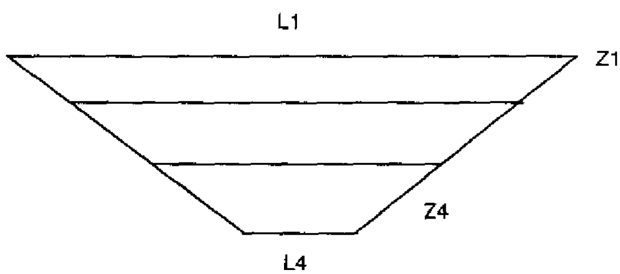
Figure 5.2: A sample cross-section of abscissa-elevation type.



5.1.6 Width-elevation

Another way to describe a cross-section **is** to enter, for each value of elevation, the *section* width as drawn in Figure 5.3. The width-elevation **couples** (Zi, Li) are entered, starting from the bottom or *top*. This description ***is*** generally adopted when precise information **of** the section **is** not available.

Figure 5.3: A sample cross-section of width-elevation type.

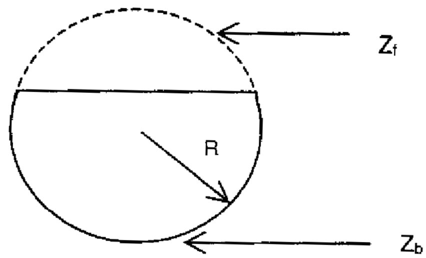


5.1.7 Parametric Form

The section of geometric shapes can be input in parametric form.
The circle is defined by its radius, R, the bottom elevation by Z_b and bank elevation by Z_f,

$$Z_f = Z_b + 2R$$

Figure 5.4: A circular cross-section in parametric format.

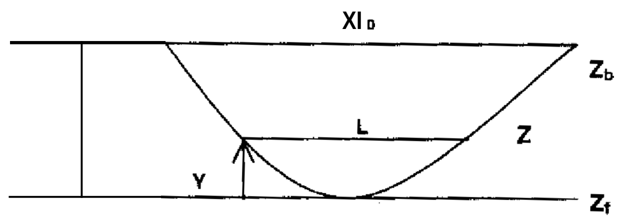


Culverts are defined by the bed-width, L, the side slope, m, bed elevation, Z_b and bank elevation, Z_f

Power relations could be used for parabolic, rectangular, trapezium or triangle sections. A parabolic section is defined as:

$$L = X_{L0} * (Y/Y_0)^a$$
$$Y_o = Z_b - Z_f$$
$$X = Z - Z_f$$
$$a = 0.5$$

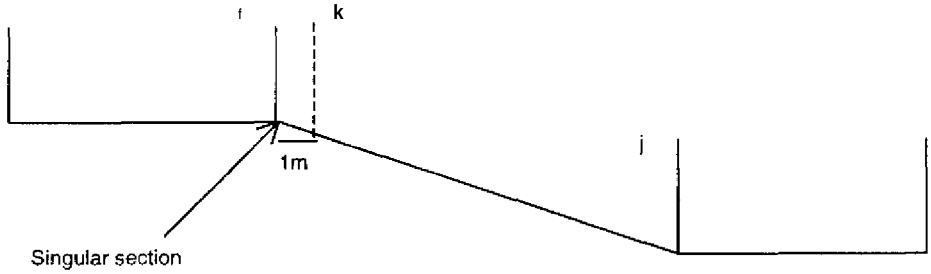
Figure 5.5: A Parabolic cross-section in parametric format.



5.1.8 Singular Section

The cross-regulators and weir in the main canal are defined as singular sections within a reach. The advantage is to have accurate water levels and flow conditions for the structure. Two cross-sections are required for the simulation of the structure and it is advisable to enter two cross-sections, upstream and downstream of the structure, at the same abscissa, especially if there is a change in geometry. Otherwise, the next section downstream is used to interpolate a section at one meter downstream of the singular section. Figure 5.6 shows an example of the interpolated section; k is the interpolated section between i and j. In the case of a drop downstream of the structure, interpolation can give wrong results.

Figure 5.6: Cross-structure as a singular section in a canal reach.



5.1.9 Output of the Topographic Unit

The topographic unit process geometric data for the steady and unsteady state simulations. **The** numerical and graphic results give longitudinal and cross-sectionai profiles, canal width, depth, perimeters and reach volumes for each computational section. *The access to this unit **is** only through **a hardware key, without** which it could not be operated.*

5.2 UNIT 2 - STEADY FLOW MODULE

Unit 2 computes the water surface profile in a canal under steady flow conditions. This water surface profile can be used as the initial condition for the unsteady flow computation in Unit **3.** The steady flow calculations allow testing the influence of modifications to structures or canal maintenance.

A sub-module of the steady flow module computes off-take gate openings to satisfy given target discharges. Another sub-module computes the cross-regulator gate openings to obtain a given target water surface elevation upstream of the regulator. To calculate **the** water surface profile **under** sub-critical steady flow conditions in a reach, the following classic hypotheses of uni-dimensional hydraulics **in** canals are considered:

- The flow direction is sufficiently rectilinear, *so* that the free surface in **a** cross-section could **be** considered horizontal.
- The transversal velocities are negligible and **the** pressure distribution is hydrostatic.
- The friction forces are taken into account through the Manning-Strickler coefficients.

For the actual canal operations, it is emphasized that the steady **flow** regime represents the objective to **be** attained, and the unsteady flow model indicates **how** best to reach these objectives.

5.2.1 Differential equation of the water surface profile

The equation of the water **surface** profile in a reach can be written as follows:

$$\frac{dH}{dx} = -S_f + (k-1)\frac{qQ}{gA^2} \qquad (1)$$

g = gravitational constant
n : Manning coefficient
R : hydraulic radius (m)

$$s_f = \frac{N^2 Q^2}{A^2 R^{4/3}}$$

A : cross-section area (m^2)
H : total head (m)
q : lateral inflow (q > 0, k = 0) or outflow (q < 0, **k = 1), (m^2/s)**
S_f : linear head losses ($m^{2/3}/s$)
Q : discharge (m^3/s)

To solve this equation, upstream and downstream boundary conditions and hydraulic roughness coefficient along the canal should be known. The equation does not have **an** analytical solution, (in the general case) and is discretized in order to obtain a numerical solution. The water surface profile is integrated, step-by-steps, starting from the downstream end.

Integrating equation between sections i and j:

$$\int_j^i dH + \int_j^i -\frac{qv}{gA}k dx + \int_j^i S_f dx = 0$$

$$H_j - H_i - \frac{kq dx_{ij}}{2g}\left(\frac{V_j}{A_j} + \frac{V_i}{A_i}\right) + \frac{S_{fi} + S_{fj}}{2} dx_{ij} = 0 \qquad (2)$$

Integrating equation [2] **between i and j**:

$H_i(Z_i) = H_j + dH(Z_i)$

A sub-critical solution exists if the curves $H_i(Z_i)$ and $H_j + DH(Z_i)$ intersect.
For this, it is **necessary** that:

$\delta = H_j + DH(Z_{Ci}) - Hi(Z_{Ci}) > 0$

Z_{Ci} is the critical elevation defined at i by:

$$\frac{Q_i^2 B_i^2}{g A^3_i} = 1$$

$\delta > 0$: Sub-critical solution, and
$\delta < 0$: Supercritical solution.

The function $f(z_i)$ does not have a continuous first derivative, because the geometry *of* the computational section is known only at certain points. Hence, Equation $f(z_i)=0$ is numerically solved using Newton's method, which uses a bisection algorithm for 'computation convergence.

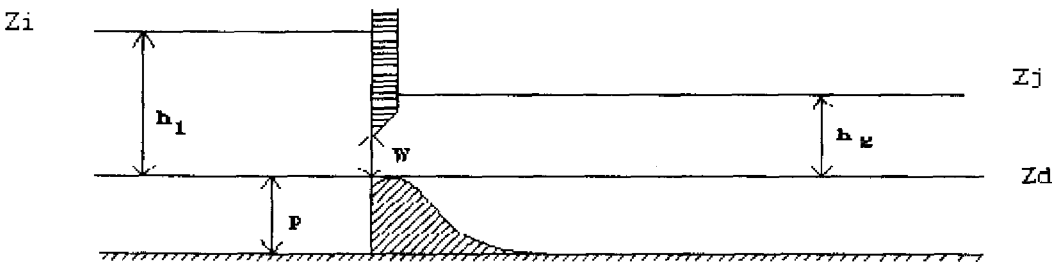
5.2.2 Cross-structure Equations

When cross-structures exist on a canal (defined as a singular section) the water surface profile equation cannot be used locally to calculate the water surface elevation upstream of the structure. The hydraulic laws of **the** different structures must be applied. The modeling of these devices is handled in many different ways by open channel mathematical models, because:

- the equations used for the hydraulic devices are many, and do not cover all possible operating conditions;
- in most cases, the range of discharge coefficient needs to be determined empirically through field experiments; **and**
- in particular, **it** is difficult to maintain the continuity between the different flow conditions, for example, at the instant of transition between free-flow conditions and submerged conditions, or between open-channel conditions and pipe-flow conditions.

In **SIC,** a set of mathematical equations that assumes a smooth transition of discharge computations for a transition in flow conditions is selected. **A** distinction has-been made between devices with a high sill elevation (Weir or Orifice) and devices with a low sill elevation (Weir or Undershot gates).

figure 5.7: An undershot orifice structure.



The following equations are **used** to compute the discharge through a structure:

5.2.2.1 Weir / Undershot Gate (Small Sill Elevation)

The standard discharge equation:

$$Q = \mu_f L \sqrt{2g} h^{3/2} \qquad (1)$$

The free flow weir equation is the same as the standard discharge equation where μ_f is the discharge coefficient, L the width of the structure and h_1 the upstream water level from the crest.

For submerged flow, the equation changes to:

$$Q = \mu_f k_f L\sqrt{2g}h_1^{3/2} \qquad (2)$$

where k_f is the coefficient of the reduction for submerged flow.

Undershoot Gate - Free Flow:

$$Q = L\sqrt{2g}\left\{\mu h_1^{3/2} - \mu_1(h_1 - W)^{3/2}\right\} \qquad (3)$$

Undershoot Gate - Submerged:

$$Q = L\sqrt{2g}\,(k_F.\mu.h_1^{3/2} - \mu_1(h_1 - W)^{3/2}) \qquad (4)$$

Totally Submerged Flow:

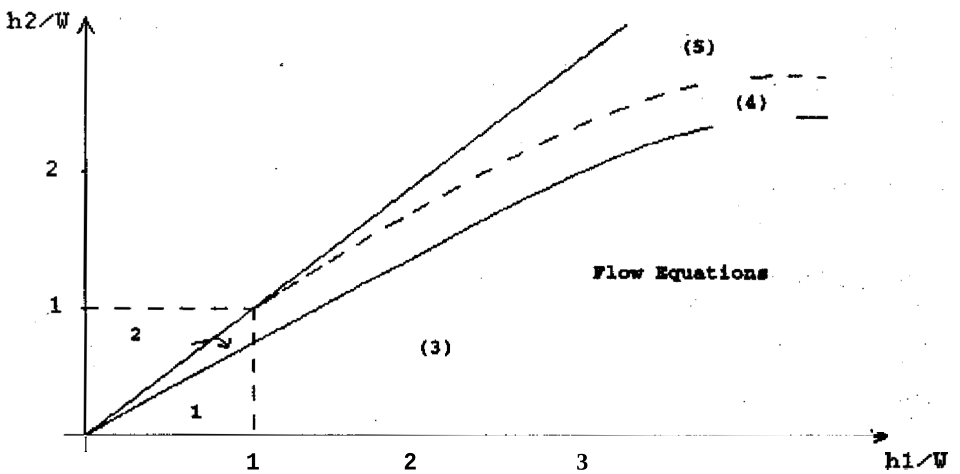
$$Q = L\sqrt{2g}\,(k_F.\mu.h_1^{3/2} - k_{F1}.\mu_1(h_1 - W)^{3/2}) \qquad (5)$$

Where k_F and k_{F1} are the coefficients computed by the model to refine the flow under different conditions. In all cases, an equivalent free flow equation is determined and entered by the user

$$C_F = \frac{Q}{L\sqrt{2g}W\sqrt{h_1}}$$

The functioning of a weir/undershot gate device is represented in the Figure **5.8** below, showing the application range of the above-mentioned equations.

Figure 5.8: The application range of *flow* equations.



5.2.3 Discharge Computations for an Off-take

To compute the precise water delivery through an off-take under steady flow conditions, the secondary canal should be described as a part of the looped model. Whereas, for the main canal model, the program is able to calculate the corresponding off-take gate opening, knowing the off-take target discharge for the following three types of downstream conditions at the head of the secondary canal:

1. Constant downstream water surface elevation.
2. Downstream water surface elevation, Z_2, that varies with the water surface elevation upstream of a free-flow weir:

$$Q(Z_2) = L\sqrt{2g}(Z_2 - Z_D)^{3/2}$$

3. Downstream water surface elevation that follows a rating curve of the type:

$$Q(Z_2) = Q_0\left(\frac{Z_2 - Z_D}{Z_0 - Z_D}\right)^n$$

The equations used are the same as given in the previous section.

5.3 UNIT 3 - UNSTEADY STATE FLOW MODULE

Unit 3 computes the water surface profile in the canal under unsteady flow conditions using Saint-Venant's equations. The initial water surface profile is provided by Unit 2 (steady flow module). The unsteady state module allows studying the transition from one operational state or schedule to another. Further, it helps to understand the behavior of canal reaches and structures under transitional conditions. A flow profile is developed over space and time, which simulates the gradually varied unsteady state flow caused by a change in inflow, or the structure's operations.

It calculates the off-fake discharges when knowing the off-take openings. But unlike Unit 2, it is not possible to automatically compute a regulator gate opening when the upstream water level is fixed, i.e. water levels are always computed by the model.

5.3.1 Saint-Venant's Equations

Assumption:

- **The** canal is divided into homogenous zones,
- only smooth transient phenomenon are considered, and
- propagation of **a** surge cannot be simulated.

5.3.2 Continuity Equation

The equation accounts for the conservation of the **mass** of **the** water **by** taking into **account the** inflow water **mass,** outflow water mass and the change in storage. The **simplified** equation expressing the conservation of mass is written as:

$$\frac{\partial A}{\partial t}+\frac{\partial Q}{\partial x}+q=0$$

where **A is** the area, such as A=Q/V, q is the supplementary discharge per unit of length, **and** ∂t and ∂x are the depth and time steps.

5.3.3 Dynamic Equation

For simplicity, the unsteady flow *is* treated as a two-dimensional steady **state** flow, while **an** additional variable for the time element is introduced, This time variable accounts for the variation in the velocity, and hence, brings in acceleration, which produces force and **causes** an additional energy loss in the flow.

The general dynamic equation for gradually-varied unsteady state flow is written **as:**

$$\frac{\partial Q}{\partial t}+\frac{\partial Q^2/A}{\partial x}+gA\frac{\partial Z}{\partial X}=-gAS_f+kqv$$

where S_f **is** the friction slope, which is computed using the Manning **formula** (explained in **the steady** state flow profile relations).

These continuity **and** dynamic equations for gradually-varied flow were first published **by** Saint-Venant. The exact integration of these equations **is** not possible; for practical applications, **a** solution of the equations may be obtained **by** approximate step-methods, or **by** simplifying assumptions.

5.3.4 Semi-implicit Discretization

A four-point semi-implicit scheme, known as the Preissman's scheme, is used to numerically solve the Saint-Venant equations. The equation is discretized **by** replacing **partial** derivatives with finite differences. Also, each reach is transformed into a **series** of n computational cross-sections connected to each other **by** two linear equations. **This** gives **2 (n-1)** equations in discharge **and** elevation. The two missing equations for the system resolution are provided **by** the upstream and downstream boundary conditions. **And,** finally 2n **equations** are solved using the double sweep method; details are not discussed here, but a reference is **made** as Part **II** of the manual of the SIC model, " THEORETICAL CONCEPTS".

Chapter

6

6. STEADY STATE HYDRAULICS OF THE DESIGN AND OPERATIONS

This section presents the steady state behavior of the canal simulated for the design assumptions. The calibration procedure of the model is explained, the sensitivity of the hydraulic parameters is tested, and the operational flexibility and constraints of the control structures are evaluated.

6.1 CALIBRATION OF THE MODEL

The calibration of a hydraulic model for the design conditions of a canal is relatively simple. The model is expected to produce an output closely matching the computations of the design procedures using the same input parameters and assumptions. Nevertheless, a steady state model compiles all canal reaches and structures designed in pieces by different persons, treats them as a unit and delineates their combined behavior. The calibration of a model coefficient becomes important if mathematical relations used by the model are different from the design relations, or if different assumptions are not coherent.

In the case of the CRBC, the lined section is designed using Manning's equations, while the geometry of the unlined section has been computed using Lacey's formula. In the hydraulic model of SIC, only Manning's equation is available, hence, an equivalent Manning's formula needs to be derived for the unlined section. A value of $n = 0.023$ is computed against a silt factor of $f=0.97$ used for the unlined reach of the CRBC.

As the model input, geometric dimensions, roughness coefficient and seepage losses are taken from the design of Stages I, II & III, while the geometry of the unlined section is taken from a survey carried out in 1998 by WAPDA.

The first step of model-building is to define the canal layout and then to enter the canal cross-section, while specifying the location of turnouts and on-line canal structures (cross-regulators, weir, siphons etc.). Once the topographic computations are complete, hydraulic data are entered in the data interface of the steady state module. At this stage, the dimensions of all structures need to be defined. Before proceeding for the analysis, the model is calibrated against a reliable data set. After achieving a reliable calibration, the model could be used to simulate steady state, as well as unsteady state scenarios.

The computed and design water levels for the full supply discharge are shown in Figures 6.1a & b. In the unlined section, computed water levels are slightly lower than the design water levels (Figure 6a), which was expected because the actual bed level is still lower than the design bed level. In the lined section, computed water levels are 10-30 cm higher in a few reaches of Stages I and II. The maximum difference in levels is upstream of two cross-regulators of Stage II. Generally, a good match between computed and design water levels represent a good calibration of the hydraulic model for the design conditions of all three stages of the CRBC.

Figure 6.1a: Water surface levels at full supply calibration of CRBC Stages I & II.

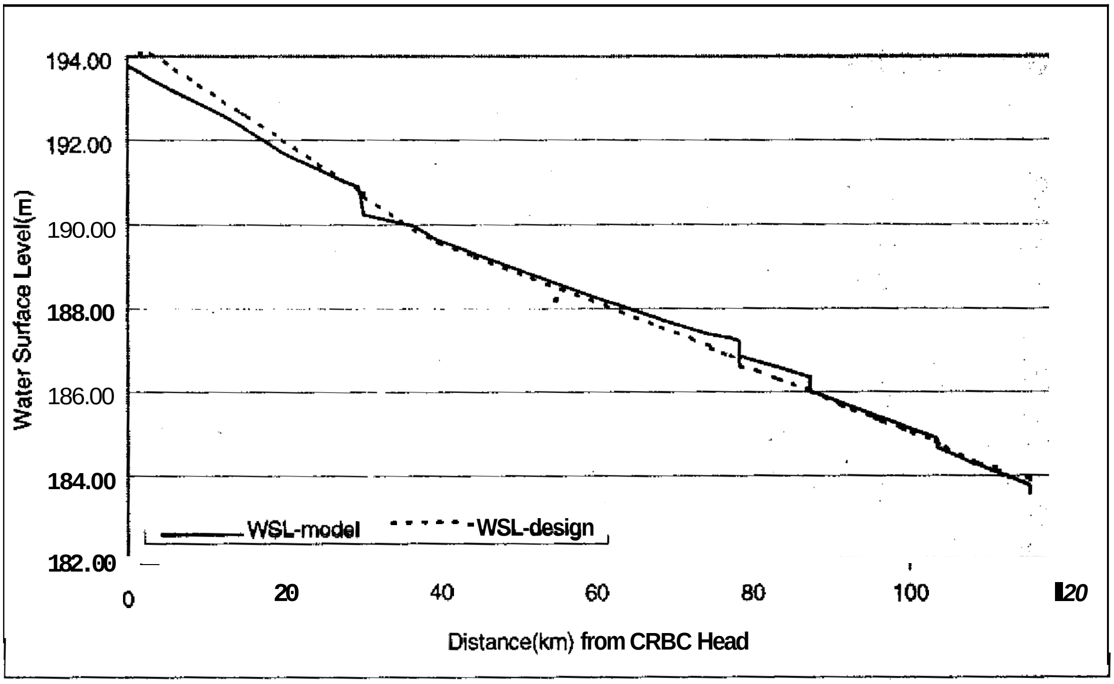
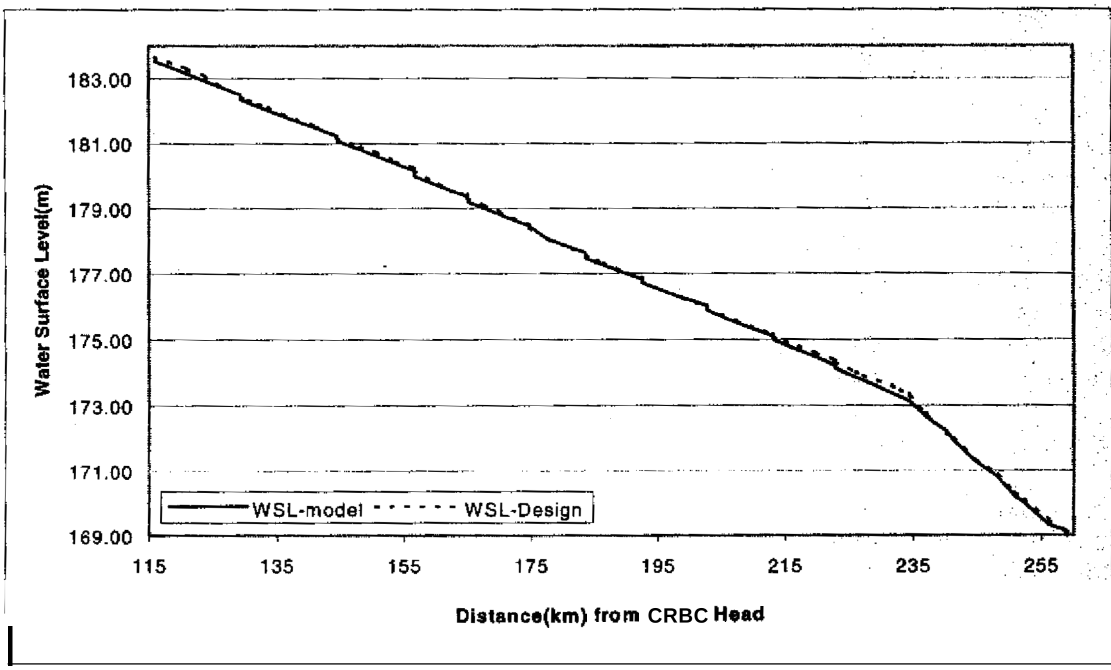


Figure 6.1b: Water surface levels at full supply calibration of CRBC Stage III.



6.2 MAIN CANAL CAPACITY

6.2.1 Design Capacity

The calibration computations at the full supply discharge does not indicate any serious difference in water levels, hence, no capacity problem is expected if the design assumptions hold good after the construction and operations of the canal. A sizable freeboard of 2 to 4 ft has been provided, which could easily cushion the normal fluctuations and instabilities within a range of 20 percent.

However, current water levels in Stages I & II proclaim that sediment deposition and maintenance conditions of the CRBC can seriously reduce the canal capacity, especially upstream of the cross-regulators and bridges. A detailed monitoring of the sediment load in the CRBC inflow for a couple of years has determined that the water diverted from the Chashma Barrage would always include a substantial amount of silt toad (ISRIP 1988-93).

6.2.2 Potential Capacity Problem

To address the CRBC capacity question with reference to the existing experience, two situations are evaluated:

1. The reduction in canal capacity in the case of a uniform deposition on the canal bed *(sensitivity scenario);* and

2. The functional capacity of Stages I and II under current operational conditions of 1998 *(learning from the* experience).

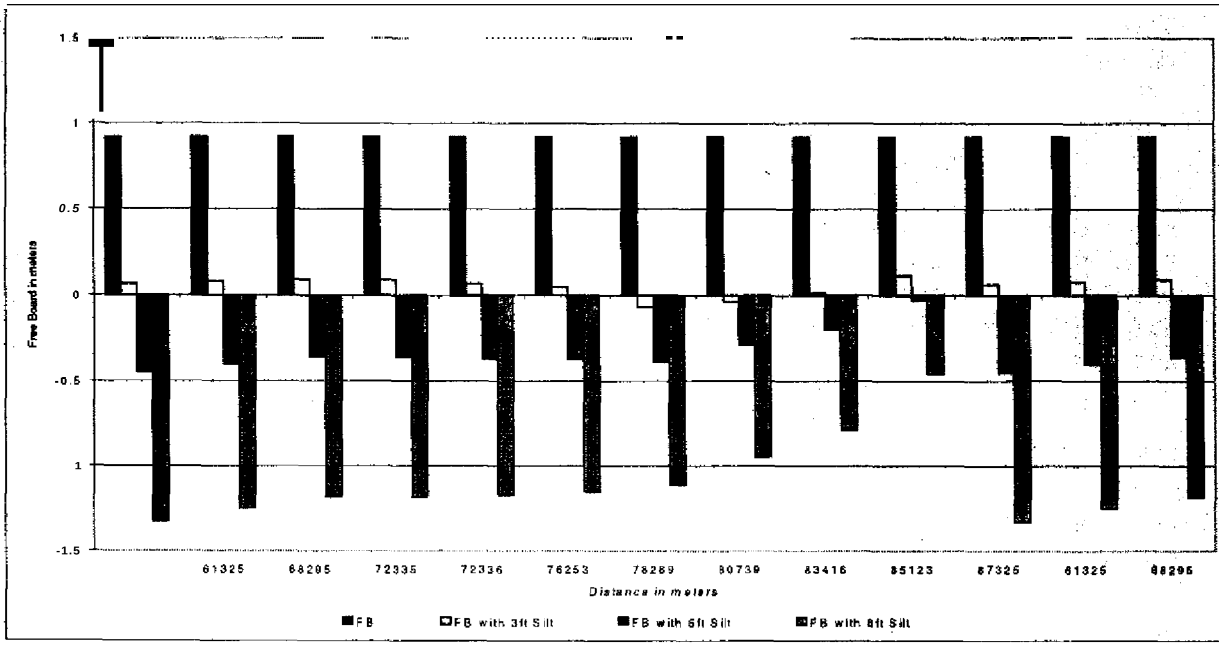
6.2.2.1 Canal Capacity in Case of a Uniform Deposition

This scenario is developed to access the sensitivity of the canal capacity for a uniform reduction in the cross-sectional area. The canal bed is uniformly raised to reduce the total canal depth by 20%, 30% and 50%. In each case, a steady state simulation is carried out for 100% inflow. The expected reduction of the capacity in the head reaches is shown in Figure 6.2. It is clear that more than 20% loss of the canal depth is serious for the canal because the remaining free board, under ideal conditions, will be less than 30 cm (1 ft), which could not absorb the expected fluctuations. At 30% reduction, the freeboard is already negative. The 50% reduction is an extreme case. The reduction along the canal shows that a higher margin is available upstream of the cross-regulators,

Theoretically, the uniform deposition of sediment is less threatening to the overall capacity of a reach or a structure, but a situation very unlikely to happen in the real field conditions. The operational conditions of a canal like the Chashma cause a variable velocity profile and a substantial velocity drop upstream of the cross-regulators. Uniform sediment deposition is not an observed or **expected** situation in these canals. The capacity problems due to sediment depositions occur in sections with low velocities and obstructions.

Nevertheless, the simulation of uniform deposition warns that a reduction of depth exceeding 10% is alarming. The limit can reach 20% only if no other maintenance problems exist and big discharge fluctuations are not expected in the canal.

figure 6.2: Sensitivity of the canal capacity to the uniform silt deposition.



6.2.2.2 Functional Capacity of Stages I & II in June 1998

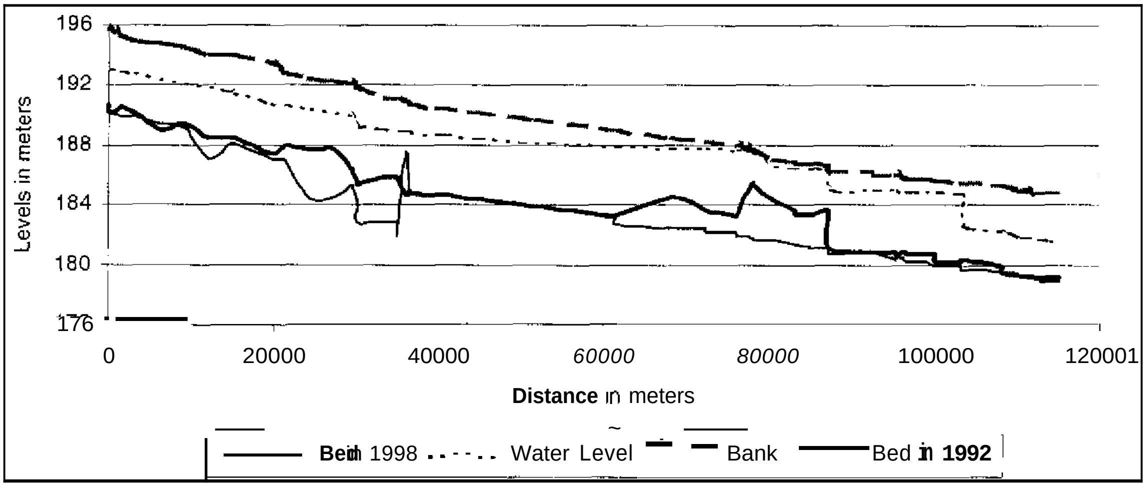
The cross-sections of Stages I & II, taken by **WAPDA** in June **1998, indicate** a sediment deposition of 1 to **4** meters in the tail reach of Stage **I** and head reaches of **Stage II. The** deposition **is** maximum upstream **and** downstream of the siphon at the and **of Stage I. The** simulation results show that under these conditions, only 79 curnecs ***(2,800*** cusecs) of water would be conveyed through Stages I and II, while consuming all of **the** free **board. The** computed water **levels** upstream of **cross-regulators are** higher than **the** design level. Raising **the** lining has already been carried out in two reaches **of Stages I** & **II.**

To accommodate monitoring inaccuracies **and** an expected bias created **by very** uneven cross-sections, it **is** assumed that the actual deposition is 25% **less** than the monitored deposition. For the **second** simulation, **4** meters (13 feet) of silt deposition is reduced **to 3 meters** (10 feet). This **allows** 85 cumecs (3,000 **cusecs)** of water to **pass** through the canal, a reduction of **40%** in the original canal capacity. The **levels, at** 3,000 **cfs,** are **shown in** Figure 6.3. The massive and localized sediment has reduced the canal capacity in two stretches of **5** to 7 km each in length. The inftuence of operations **is** obvious, **as** there **is** exceptionally high silt deposition upstream of the cross-regulators.

The unusual **formation** of sediment heaps along the **right** bank of the main canal opposite ***to*** the distributary head regulators might already be affecting their functioning by passing **on an** extra **silt** load to the secondary canals. If allowed to **be** continued for a longer period, **the** maintenance requirements of the **secondary** system may increase substantially.

The nature of the present sediment deposition is local, caused by a constant heavy water ponding upstream of the cross-regulators of Stages I & II. Moreover, field visits indicate that the maintenance conditions of the main canal are not good. Lining deterioration and small sediment deposition occur in most of the reaches. The original capacity of the canal can only be restored with 100% silt clearance and the restoration of the designed canal prism without any damage.

Figure 6.3: Bed and water levels of Stages I & II in 1992 and 1998, at 67% head discharge.



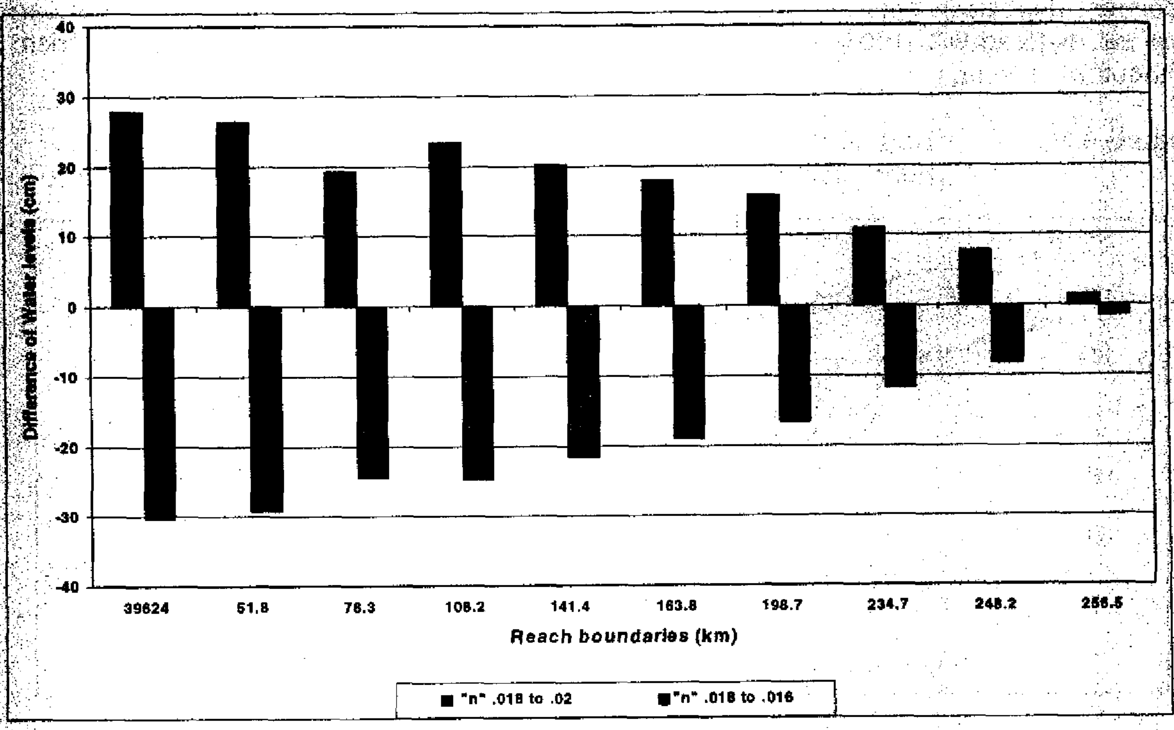
6.3 SENSITIVITY TO THE ROUGHNESS COEFFICIENT

The value of roughness in a lined reach depends upon the type and quality of lining as well as the actual conditions of the canal prism, like silt deposition, weed growth, etc. Most of the canals, after their construction and operations, indicate higher than the originally assumed values of roughness. A previous study of IIMI (1992) has indicated an increase of roughness in Stage I due to sediment deposition. For a canal where the operational velocity is less than the design velocity, the computations for roughness could be biased due to a change in cross-sectional area or backwater influence. The appropriate method to compute a roughness coefficient is by simulating an ideal reach with all measured parameters operating under free flow conditions.

Two values of 'n', 0.016 and 0.018, have been used for the design of Stages I & II and Stage III, respectively. To assess the impact of roughness changes, water levels are computed for **three** values of n, 0.016, .018 and 0.02, for the whole CRBC. Figure 6.4 shows the difference in water levels in each reach caused by this variation, taking .018 as the **base** case. The maximum difference is 30 cm in the maximum capacity reach, and decreases with the canal depth towards the tail.

An increase in actual roughness after the construction decreases the available freeboard and increases the working head at different structures, while the lower actual roughness may cause decreases in the available working head of the off-taking structures.

Figure 6.4 Average water depth changes in CRBC reaches due to a change in roughness coefficient.



6.4 STEADY STATE WATER LEVELS AND VELOCITIES AT DIFFERENT FLOW RATES

According to the Water Apportionment Accord, the CRBC would be operated at 30% to 100% of its full supply on ten-daily bases (see Section 3). The corresponding water depth in each reach of the canal vary substantially if the cross-regulators are not operated. The required operations would depend upon the targeted schedule of operations and operational flexibility of the system. This section computes the water level for two basic situations, which would be further used in the proceeding sections to evaluate different operating scenarios:

- a uniform and proportionate distribution of water along the CRBC main canal (*without operations*); and
- a proportionate delivery of water to the secondary system (*with operations*).

6.4.1 A uniform and Proportionate Distribution of Water along CRBC Main Canal (Without Operations)

The water levels are computed for the authorized flow range of the CRBC under steady state hydraulic conditions. The uniform water profiles are developed using gradually varied flow equations. It is assumed that water is uniformly distributed along the canal at each off-taking position. A proportionate share of the discharge is subtracted from the main flow.

The computed water levels are shown in Figures 6.5a & 6.5b for the releases of 30% to 100% discharge from the Chashma Barrage. All of the cross-regulators are operated as weir structures and no backwater is allowed. These computations provide free flow water levels for the full discharge range. The difference of the depth between the minimum and maximum discharges is from one to three meters. This is the depth of water, which would be managed through operations.

For the upstream control irrigation systems of India and Pakistan, the recommended flow range is 80% to 110% of the design discharge. Within this range, it is possible to achieve a proportionate supply to the secondary system and convey a uniform flow to the reaches downstream.

6.4.2 A Proportionate Delivery of Water to the Secondary System (With Operations)

The previous scenario is further developed to compute the required water levels and gate operations for the proportionate distribution of conveyance and delivery at the selected flow rates. This scenario provides another baseline set of information by computing backwater and gate openings while maintaining the appropriate water levels in the main canal.

The water levels in Stage I, II & III at 30, 43, 67 and 100% of the design discharge are shown in Figures 6.6a and 6.6b. For each case, efforts are made to attain the minimum backwater to feed the farthest distributary. These optimal operating conditions represent ideal operations of the main canal, which could only be implemented if a variable distribution of water could be managed at the secondary level.

At 100% head supply, more than 95% off-takes can draw their proportionate full supply without any operation of the cross-regulators. In two reaches of Stage II, cross-regulators are operated to feed distributaries with higher than design water levels.

At 67% of the full supply, cross-regulators of Stages I & II need to be operated to feed some of the distributaries by 67% of their design discharge. No ponding is required to deliver a proportionate supply within Stage III.

At 43% of the discharge, substantial ponding is required to feed the secondary canals of Stage I & II, while Stage III off-takes can draw their proportionate share with the operations of the cross-regulators.

For proportionate deliveries at 30%, water levels upstream of cross-regulators of Stages I & II are almost at the full supply level, which means that to feed the distributaries at 30%, water levels upstream of the cross-regulators would be raised to the 100% level, i.e. to a maximum storage depth of 3 meters.

Figures 6.6a and 6.6b also indicates a difference in the cross-regulators' behavior. At 30% and 43% of the discharge, some of the cross-regulators have to maintain a working head for the farthest distributary immediately downstream of the upstream cross-regulator. If the upstream cross-regulator is operating under submerged conditions, both regulators will influence each other. At 43% of the proportionate distribution, four consecutive cross-regulators of Stages I & II are under the influence of each other. If an operation is carried out on one of them, all of them will be influenced and if operated in a response, will further affect each other. A single sizeable operation can induce an instability affecting many structures and could be continued for a long time, i.e. more than a day.

Figure 6.5a: Computed water surface levels for the imposed uniform discharges in Stages I & II.

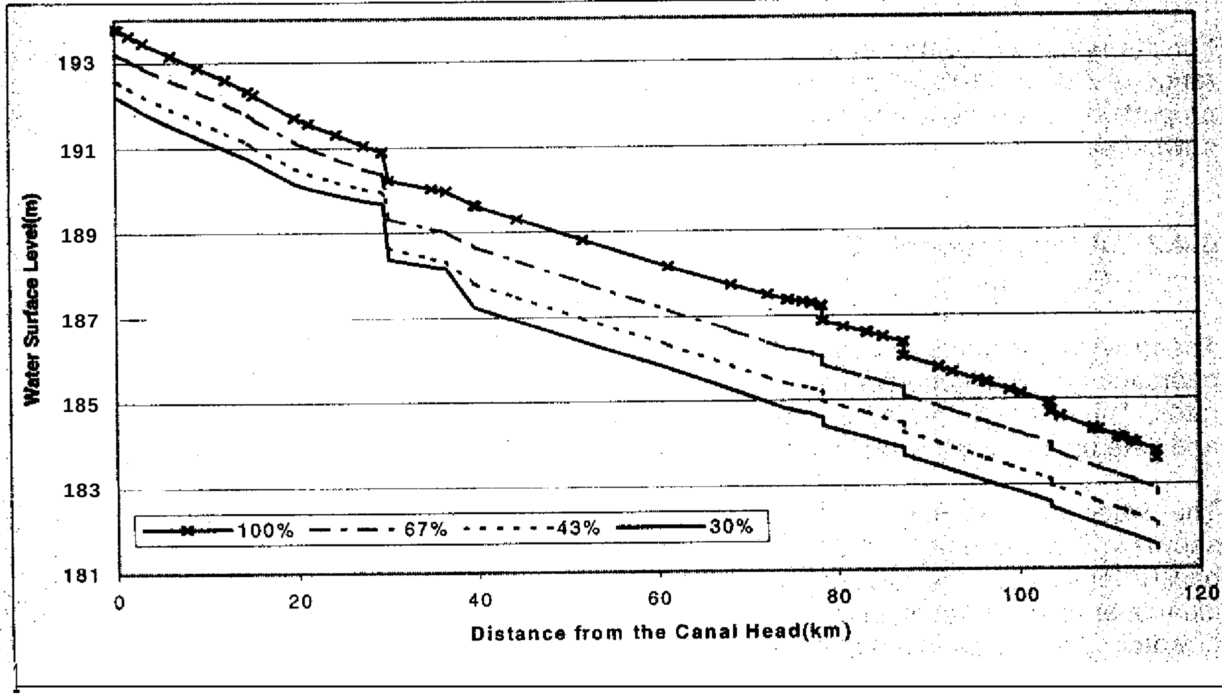


Figure 6.5b: Computed water surface levels for the imposed uniform discharges in Stage III.

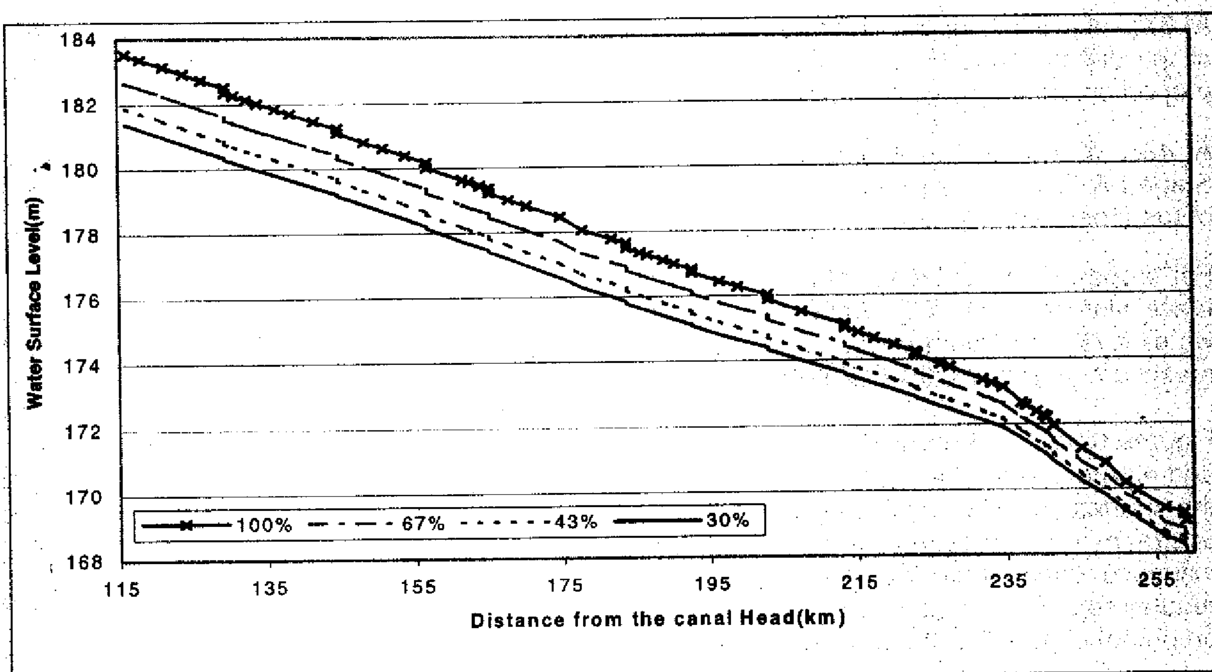


Figure 6.6a: Water surface levels for proportionate distribution in *Stages* I & II.

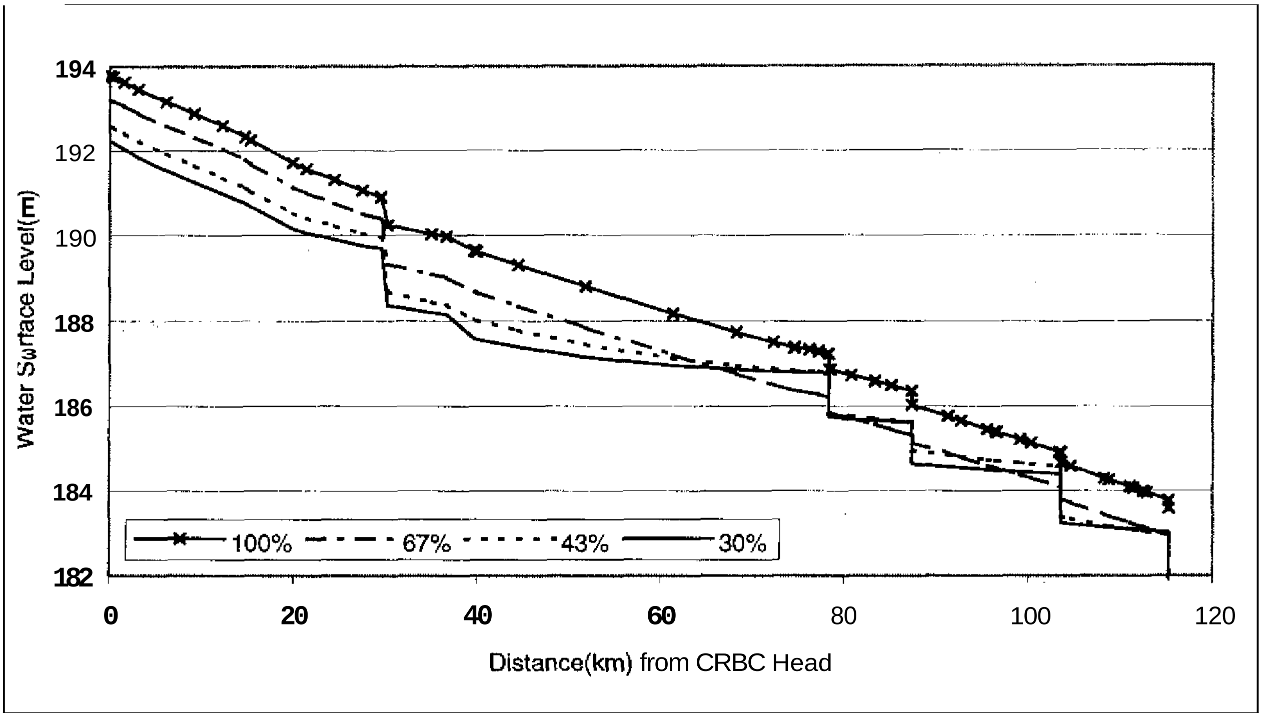
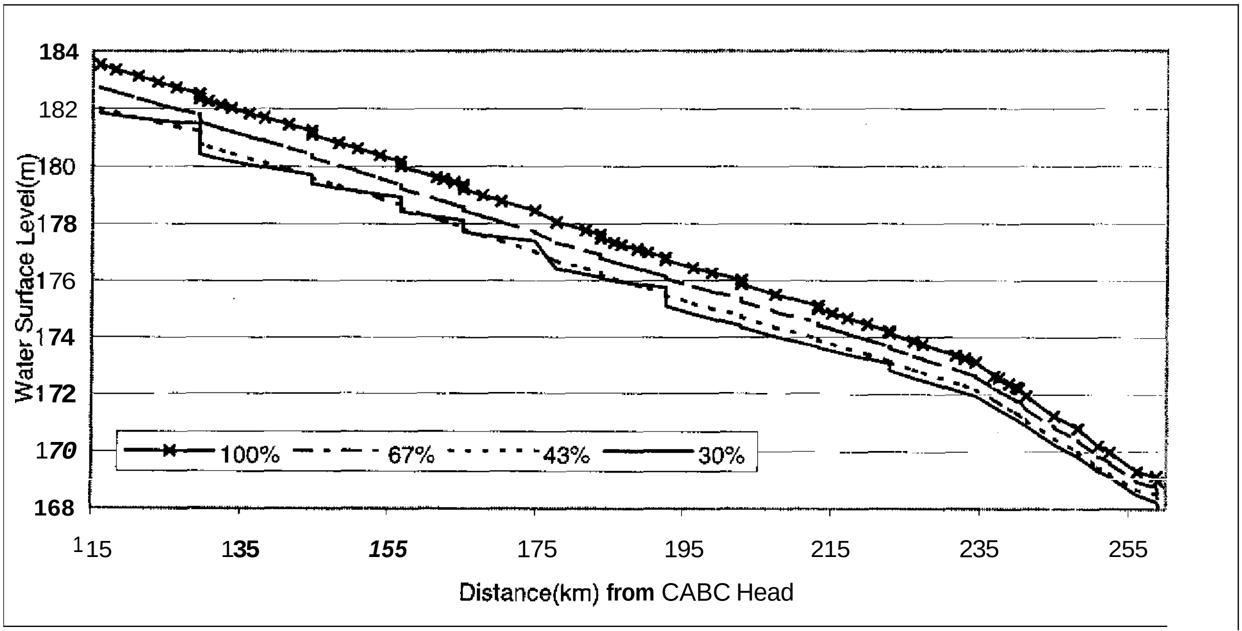


Figure 6.6b: Water surface levels for proportionate distribution in Stage III.



6.5 BEHAVIOR OF VELOCITY UNDER DIFFERENT OPERATIONS

To maintain a uniform velocity in a conveyance and distribution canal is never possible. For variable flow canals where reach storage is an operational tool, velocity is not a considered parameter at the stage of design and operational planning. However, the experience of the Chashma Right Bank Canal Stages I & II and many other canals in Pakistan shows that it is essential to provide a critical minimum velocity at low flows to avoid sediment deposition and maintenance requirements.

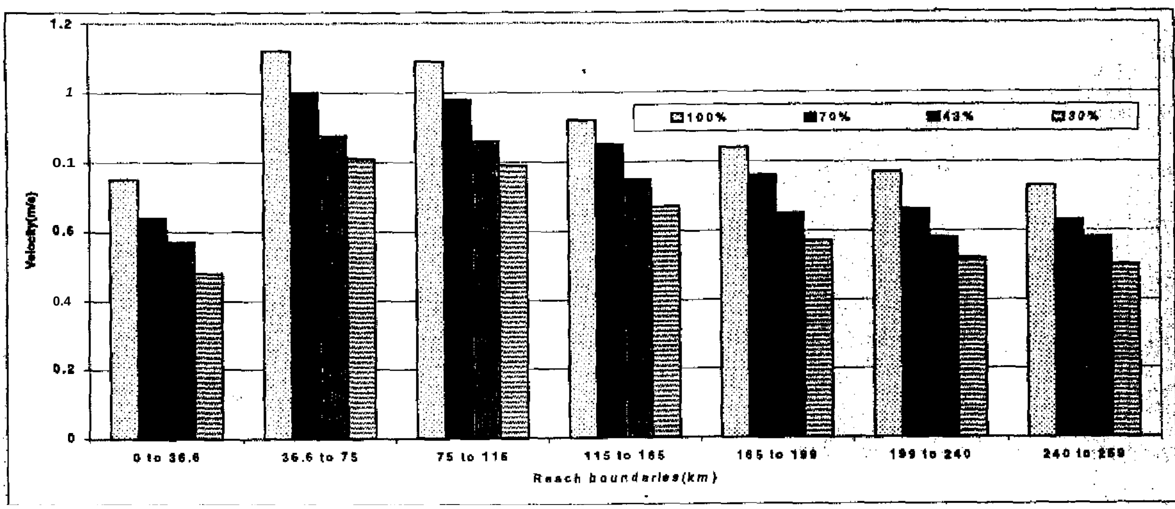
In the CRBC main canal, the velocity drops due to two reasons:

- **Head** inflow is less than the design discharge. **Cross-regulators** are operated to raise the water levels.
- The behavior of the velocity for three different operating conditions of the CRBC is **briefly** discussed below.

6.5.1 Uniform Proportionate Flow

The average reach velocity along the CRBC for the discharge range (30% to 100%) is plotted in Figure 6.7. The drop in the velocity is mild and uniform along the canal because cross-regulator has not been operated and no ponding has occurred in the canal. The velocity at the minimum flow is about 66% of the design velocity.

Figure 6.7: Simulated average reach velocity at different flow rates along the CRBC for uniform proportionate flow.

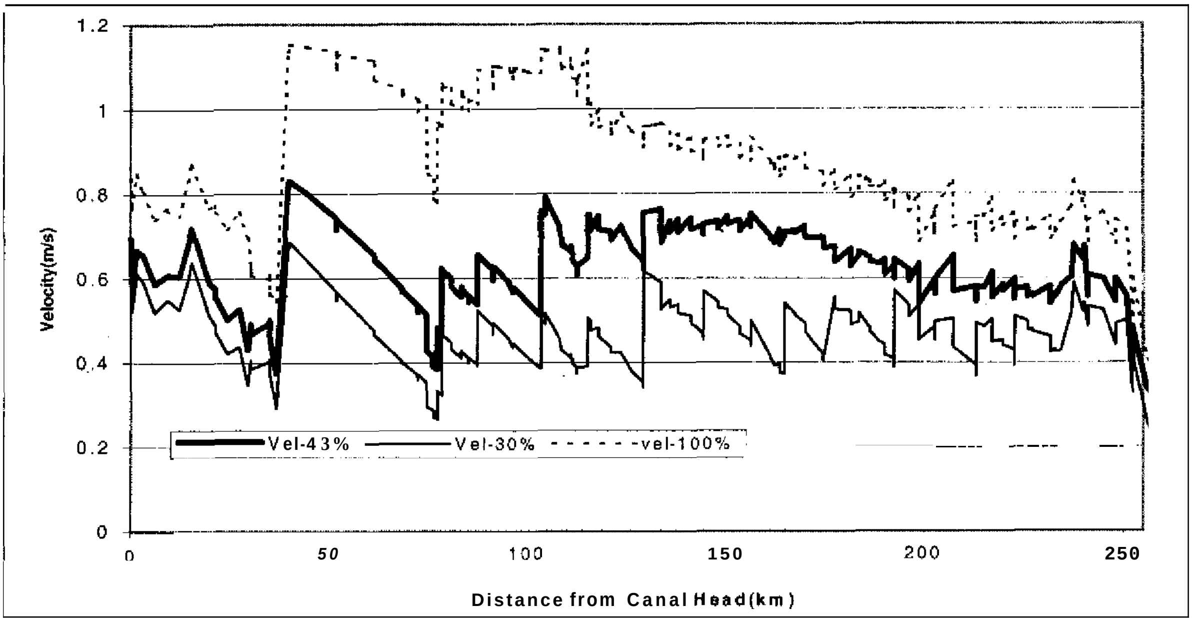


6.5.2 Operated Proportionate Flow

Figure 6.8 shows the drop in velocity at 30% and 43% of proportionate operations. The velocity is not uniform along the canal; it drops upstream of the cross-regulators depending upon the level of ponding. The velocity varies from reach to reach and along each reach.

The average velocity, in this case, is half of the velocity shown in the previous case, the minimum being only 30% to **40%** of the maximum.

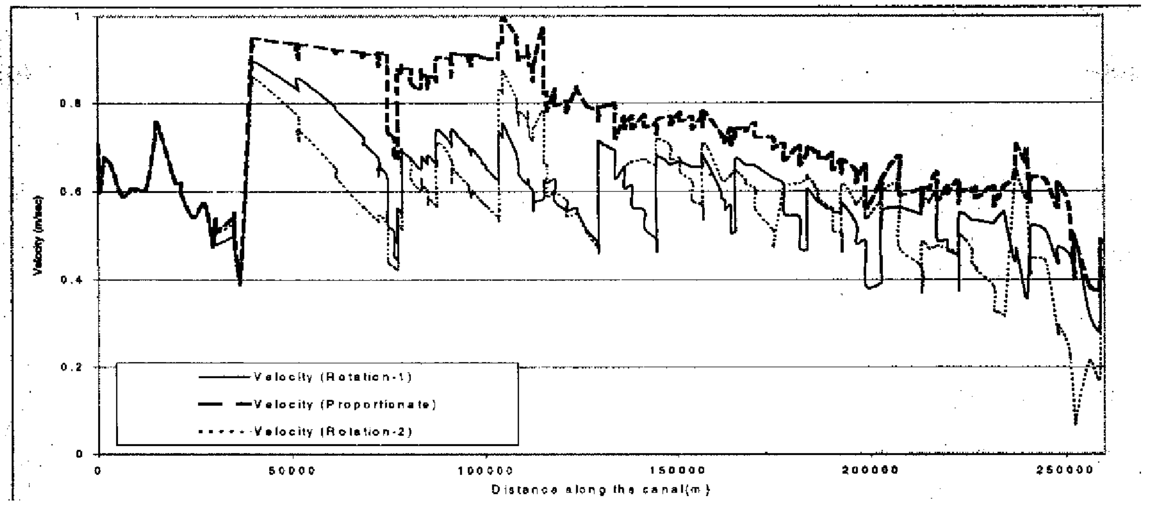
Figure 6.8: Velocity profile along the CRBC at the full supply and low flows for proportionate distribution.



6.5.3 Full Supply Rotation at *50%* Inflow

This is **the** scenario representing the planned operations of the CRBC. Figure 6.9 shows how the velocity will be affected under this type of operation. The rotation has been carried out among different reaches of the canal. The drop in the velocity is substantial. This vetocity behavior will drastically reduce **the** silt-carrying capacity of flows **and** encourage an uneven sediment deposition. The velocity analysis indicates that **a** continuous non-obstructed flow is the most favorable operating condition for the velocity, whereas the high pond levels severely affect the velocity, not only **by** causing it to drop but also by making it inconsistent.

Figure 6.9: Velocity profiles for the full supply and *50%* rotation.



6.6 WATER DELIVERY CAPACITY OF THE MAIN CANAL

The delivery capacity of a canal is different from its conveyance capacity. The conveyance is the potential of a canal to pass on the authorized discharge through the canal reaches, while delivery is its ability to deliver the maximum design discharge (or a discharge range in variable inflow systems) to the secondary system. The previous sections show that the oonveyance capacity of the CRBC main canal does not face any serious problems if the design of the physical conditions are maintained. But, it is influenced and interrupted by operations of the secondary system.

Delivery capacity of the CRBC main canal at low flows would be determined by:

- available working head for the secondary canals, and
- ability of the main canal to achieve and maintain the required working head.

Table 6 summarizes results of steady state computations in terms of water depths with reference to the crests in the main canal at different flow rates. The upstream water depth with reference to the crest for all distributary head regulators is computed for 30, 43, 67 and 100% of the discharge. The downstream full supply depth is taken from the design data. A negative depth, with reference to the crest, indicates that the water levels are lower than the crest level of a regutator. The table shows a negative upstream head at 30% of the supply for almost all of the distributaries in Stages I, II and in the first reach of Stage III. At 43% of the discharge, half of the distributaries in these reaches have negative heads, while at 50%, about one-third of the distributaries have crests above the water levels. The design water levels downstream of the distributaries head regulators are relatively high due to high command areas fed by the first reaches of the distributaries.

The design consultants of Stage III have setected the 67% water level as the critical level to deliver the full supply to Stage III distributaries. Columns 3, 7 and 10 of the table show that the working head (Ha-Hb) is positive for Stage III, but still negative for many of the distributaries of Stages I & II. The required incremental water depth can be directly computed from these data.

Table 6: Water depths, working head and the water drawing capacity of the head regulators.

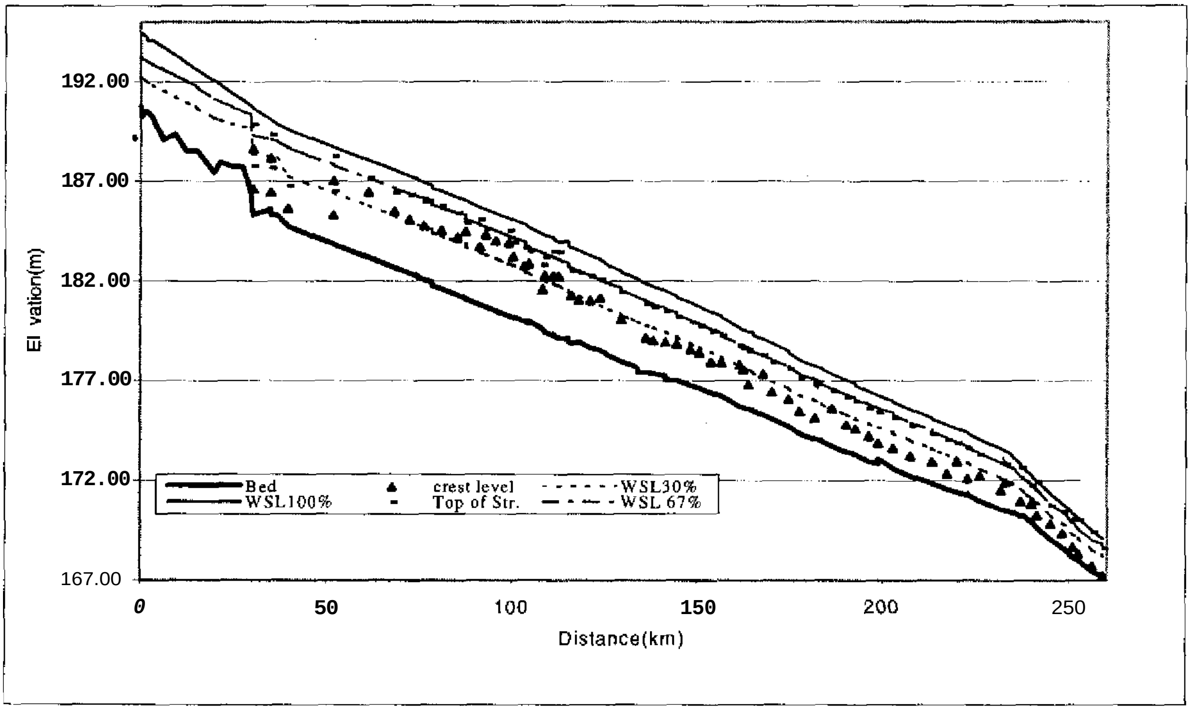
Name of Structure	Upstream Water Depth with reference to Crest (Ha)					DWS depth w.r.t. Crest Hb	Max. Discharge	Des. Off-take Discharge	Req head
	Design (m)	67% (m)	50% (m)	43% (m)	30% (m)	(m)	(m3/s)	(m3/s)	(m)
Bilot D-1 R/S								0.44	
Bilot D-2 R/S								0.10	
Bilot D-3 R/S								0.16	
Stage-I X-Regulator									
Takarwah Upper Barrel	2.10	0.69	0.19	0.01	-0.26				
Takarwah Lower Pipes	4.12	2.71	2.21	2.03	1.76		10.95	4.64	0.74
Kot Hafiz Upper Barrel	1.93	0.94	0.42	0.25	0.06		8.21		
Kot Hafiz Lower Pipes	3.63	2.64	2.12	1.95	1.76		11.79	6.43	1.08
Lining Transition									
Kathgarh Link	3.91	3.00	2.42	2.16	1.60		5.41	1.47	
Saiduwali Minor								0.12	
Link Feeder Upper Barrel	1.67	0.77	0.21	-0.07	-0.63		10.18		
Link Feeder Lower Barrel	3.40	2.50	1.94	1.66	1.10		7.26	4.11	1.09
Disty-1	1.61	0.70	0.16	-0.12	-0.67	0.94	1.10	0.71	0.53
Disty-2	2.04	1.14	0.60	0.31	-0.25	1.45	2.79	1.42	0.41
Disty-3	2.18	1.27	0.74	0.44	-0.14	1.5	7.40	3.21	0.30
Disty-4	2.23	1.38	0.86	0.55	-0.04	1.83	4.89	5.95	0.45
Stage-I Tail Reg									
Disty-5	1.94	1.12	0.57	0.28	-0.30	1.64	4.86	3.52	0.16
Disty-5A								0.97	
Disty-6	2.01	1.23	0.69	0.40	-0.17	1.71	4.89	2.55	0.08
Disty-7A	1.56	0.79	0.26	-0.03	-0.60	0.74	0.39	0.22	0.25
Stage-II X-Reg1									
Disty-7	1.98	1.12	0.55	0.28	-0.27	1.22	8.72	3.22	0.10
Disty-7B	1.26	0.39	-0.18	-0.45	-1.00	0.77	0.14	0.08	0.19
Disty-8A	1.37	0.51	-0.06	-0.34	-0.89	0.85	0.88	0.56	0.21
Disty-8B								0.14	
Disty-8	1.22	0.35	-0.21	-0.48	-1.03	0.79	0.51	0.32	0.18
Disty-9	1.82	0.96	0.40	0.12	-0.42	1.29	0.71	0.46	0.22
Disty-10	2.07	1.20	0.65	0.37	-0.17	1.36	1.47	1.01	0.34
Stage-II X-Reg2									
Disty-10A	1.78	0.80	0.27	0.01	-0.51	1.13	0.62	0.42	0.29
Disty-11	2.74	1.81	1.26	0.99	0.47	2.19	2.18	0.85	0.08
Disty-11A	2.00	1.07	0.52	0.26	-0.27	1.49	4.77	2.59	0.15
Disty-12	1.87	0.93	0.38	0.12	-0.40	1.37	4.20	1.75	0.09
Disty-13	1.73	0.87	0.32	0.06	-0.46	1.34	5.56	3.16	0.13
Stage-II X-Reg3									
Disty-14	2.37	1.39	0.86	0.61	0.10	1.24	8.37	4.11	0.27
Disty-15	2.46	1.45	0.93	0.67	0.18	1.47	7.82	3.28	0.18
Disty-16	2.25	1.24	0.71	0.46	-0.04	1.21	6.31	2.18	0.12
Disty-16A	1.91	0.89	0.36	0.10	-0.40	0.84	1.80	0.45	0.07
Direct-Outlets								0.07	0.12
Disty-17	2.41	1.57	1.03	0.77	0.26	1.30	4.14	1.50	0.15
Stage-III X-Reg1									
Direct-Outlets								0.06	0.10
Direct-Outlets								0.08	0.16
Direct-Outlets								0.06	0.10
Disty-18	2.73	1.87	1.37	1.13	0.65	1.61	6.20	2.44	0.17
Disty-18A	2.72	1.85	1.36	1.12	0.64	1.61	6.17	2.52	0.19
Disty-19	2.55	1.68	1.19	0.95	0.48	1.45	5.09	2.15	0.20
Disty-20	2.37	1.56	1.07	0.83	0.36	1.30	4.06	1.59	0.16
Stage-III X-Reg2									
Disty-20-A	2.33	1.44	0.97	0.74	0.30	1.30	3.98	1.39	0.13

Name of Structure	Upstream Water Depth with reference to Crest (Ha)					DWS depth w.r.t. Crest Hb	Max. Discharge	Des. Off-take Discharge	Req head
	Design (m)	67% (m)	50% (m)	43% (m)	30% (m)	(m)	(m3/s)	(m3/s)	(m)
Disty-21	2.33	1.43	0.96	0.73	0.28	1.30	3.99	1.47	0.14
Disty-22	2.50	1.64	1.17	0.94	0.50	1.45	4.96	2.01	0.17
Disty-23	2.33	1.45	0.97	0.74	0.29	1.30	3.98	1.56	0.16
Stage-III X-Reg3									
Disty-24	1.82	1.03	0.57	0.34	-0.09	0.84	1.72	0.51	0.09
Disty-24A	1.99	1.22	0.76	0.54	0.10	0.99	2.37	0.82	0.12
Disty-25	2.64	1.86	1.40	1.18	0.74	1.60	5.96	2.61	0.20
Stage-III X-Reg4			2.46						
Disty-26	1.75	0.92	0.48	0.26	-0.16	0.84	1.66	0.34	0.04
Disty-27	2.46	1.63	1.18	0.96	0.54	1.45	4.87	1.90	0.15
Disty-28	2.41	1.63	1.17	0.95	0.52	1.45	#VALUE!	1.67	0.12
Disty-29	2.58	1.87	1.44	1.22	0.81	1.60	5.78	2.29	0.15
Disty-30	2.61	1.93	1.49	1.28	0.87	1.60	5.89	2.49	0.18
Stage-III X-Reg6		2.54	2.11						
Direct-Outlet								0.17	0.75
Disty-31	1.73	1.06	0.57	0.36	-0.03	0.84	1.64	0.62	0.13
Direct-Outlet								0.12	0.37
Disty-32	2.24	1.52	1.10	0.90	0.50	1.30	3.81	1.59	0.16
Disty-33	2.27	1.55	1.13	0.92	0.52	1.30	3.87	1.64	0.18
Stage-III X-Reg7									
Disty-34	2.23	1.58	1.18	0.98	0.60	1.30	3.78	1.47	0.14
Disty-35	2.39	1.74	1.34	1.14	0.76	1.45	4.69	2.04	0.18
Disty-36	2.34	1.75	1.35	1.15	0.77	1.45	4.57	1.81	0.14
Stage-III X-Reg8									
Disty-37	2.34	1.68	1.32	1.12	0.78	1.45	9.11	3.68	0.14
Disty-38	2.19	1.56	1.20	1.01	0.67	1.30	3.70	1.56	0.16
Stage-III X-Reg9									
Direct-Outlet								0.12	0.38
Disty-39	2.44	1.78	1.44	1.26	0.94	1.60	5.36	2.27	0.15
Disty-40	1.63	0.95	0.62	0.44	0.13	0.84	1.54	0.51	0.09
Disty-41	2.28	1.58	1.25	1.07	0.76	1.45	4.41	1.64	0.12
Stage-III X-Reg10									
Disty-42	1.79	1.13	0.82	0.64	0.35	1.01	2.10	0.71	0.09
Direct-Outlet								0.14	0.52
Disty-43	2.09	1.37	1.07	0.91	0.63	1.30	3.50	1.30	0.11
Disty-44	1.55	0.86	0.56	0.39	0.12	0.84	1.47	0.20	0.01
Stage-III X-Reg11									
Disty-45	1.78	1.25	1.38	0.83	0.60	0.99	5.21	2.27	0.15
Direct-Outlet								0.04	0.05
Direct-Outlet								0.11	0.33
Disty-46	1.45	0.98	0.71	0.55	0.31	0.84	1.36	0.28	0.08
Stage-III X-Reg12									
Disty-47	1.74	1.26	1.00	0.85	0.62	0.99	5.08	2.18	0.14
Stage-III X-Reg13									
Disty-48	1.51	1.02	0.78	0.64	0.43	0.84	1.42	0.40	0.05
Disty-49	1.52	1.06	0.83	0.68	0.48	0.84	3.23	1.39	0.13
Stage-IIIX-Reg14									
Disty-50	1.64	1.17	0.97	0.82	0.65	0.99	3.17	1.64	0.18
Disty-51	1.75	1.59	1.09	0.94	0.79	0.99	5.10	2.52	0.19
Stage-III X-Reg15									
Disty-52	1.73	1.22	1.05	0.89	0.76	0.99	5.01	2.12	0.13
Stage-III X-Reg16									
Disty-53	1.31	0.80	0.60	0.42	0.32	0.90	6.19	2.96	0.14

6.6.1 Water Levels and Elevations of the Structures

The position of the **crest** and the tops of the regulators, along with water levels at 30% and **100%** discharges is shown in Figure 6.10. The constraint at low flows and the relative flexibility available for **each** structure is **obvious** from this graph.

Figure 6.10: Canal water levels and structure elevations (top and bottom).



6.7 WATER-DRAWING CAPACITY OF THE HEAD REGULATORS

The maximum discharge drawing capacity of distributary head regulators of the CRBC **is** shown in Column *8* of Table 6. **At** the full supply, most of the regulators **can** draw more than double the maximum authorized discharge. This flexibility has been provided to compensate the working head constraint at **low** flows. The current selection of design parameters addresses the working head problem **by** providing a higher depth at the head regulators.

The level of flexibility provided at **low** flows **poses** *a* "control **and** management constraint" at high **flows,** at the full supply only half of the gate openings would **be needed** to achieve the target discharge.

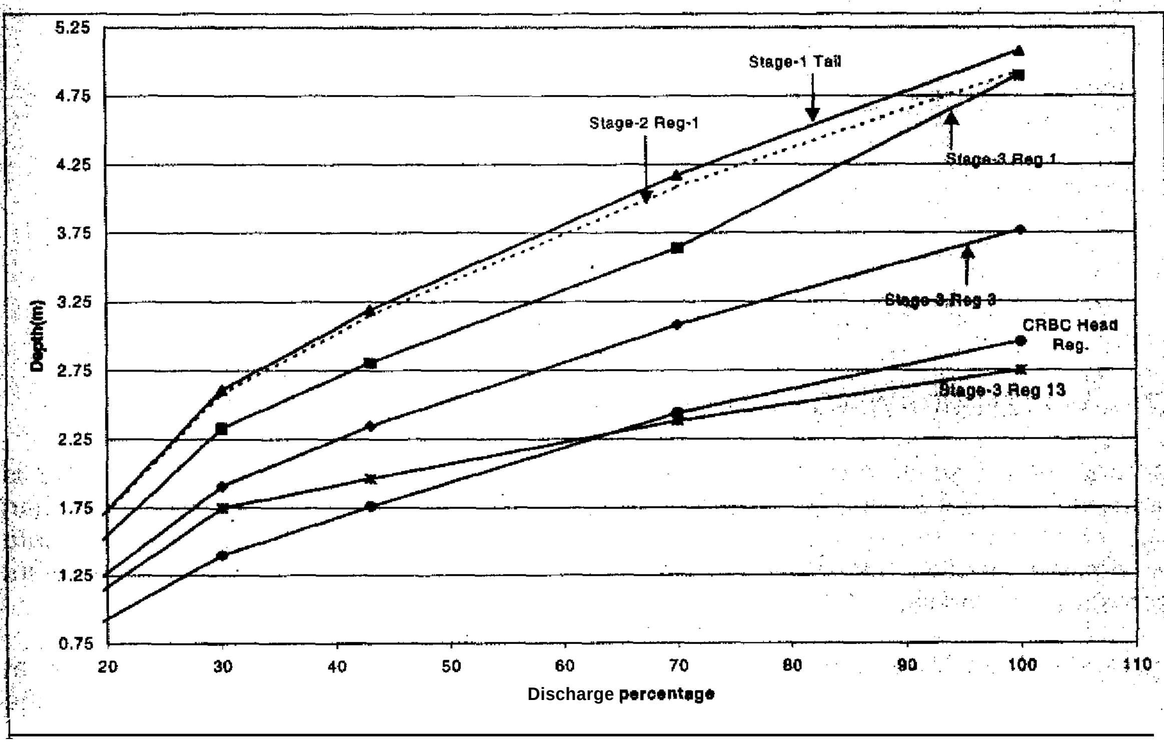
6.8 POOL OPERATIONS

The reach storage should **be** operated to minimize the water depth fluctuations in the main canal and to control supply variations to the secondary system. For the optimal utilization of storage, normally, distributaries are physically situated upstream *of* the cross-regulators, *so* that a distributary could draw water from the maximum pool without affecting the water depth and operations of other distributaries, In the case of the CRBC, distributaries are scattered all along the reaches, which could cause either undesired fluctuations of the water levels or extra storage in a reach. The second situation is currently practiced in Stages I & II. The water levels are kept fixed for different inflow-delivery patterns. An undesired consequence of this operation is low velocities causing heavy sediment deposition upstream of the regulators.

The constant depth, or fixed level operation, is an established control method for the operation of supply-based and the upstream control system. The 10-daily water allocations to the CRBC are crop-based, principally, while the canal operations are supply-based. A technical constraint is the provision of operational flexibility only in the main canal. The current silt deposition and water delivery problems suggest that the system could not work purely as a supply-based system; constraints as well as flexibility should be considered for the supply and demand sides.

A modification in the operating criteria could be introduced by adopting more than one set of target water levels upstream of the cross-regulators, for different flow ranges.

Figure 6.11: The depth-discharge curves for different structures.



6.8.1 Computation of Storage Depth at Low Flows

A simple computation example is presented here to have a gross assessment of the water depth and the time required to achieve it. The simulated water levels for the uniform and proportionate distribution (Figure 6.11) are used for these calculations.

At the minimum allocated discharges to the CRBC, all of the cross-regulators will need to be operated to feed the distributaries in their respective upstream reaches. The volume of the wedge-storage in a reach depends upon:

- distance of the farthest upstream distributary in a reach from the cross-regulator:
- maximum incremental working head required by any distributary of the reach;
- length of the reach; and
- flow conditions of the upstream regulator.

The number of secondary canals in a CRBC reach varies from ten to one, and the distance of the farthest distributary varies from 20 kilometers (Disty I of Stage I) to one kilometer (Disty 47 of Stage III). The height of distributary structures with reference to the main canal bed also varies in a big range. By utilizing the water depth computation, the depth of the storage can be roughly estimated from the difference in water levels for two particular steady states. These states could be from zero (no ponding) to a particular storage level, or two different storage levels.

A simple formula to estimate the storage depth for a particular distribution pattern and the time required to achieve this is presented below.

For practical purposes, the storage depth at the tail of a reach (upstream of the cross-regulator) is used as a reference parameter and the time required to achieve this depth is computed. The simulation of water levels with and without ponding at different flow rates are used to assess the required storage depth upstream of different cross-regulators.

The discharge-depth curves for the six cross-regulators are shown in Figure 6.11. The following example explains how these data are used to compute the required level of ponding and the time required to achieve it, at the tail of Stage I to feed Distributary I.

▪ At 43% inflow, depth without ponding	=	3 meters	(Figure 6.5)
▪ The required depth to feed distributary 1 at proportionate discharge, case-1.	=	5 m	(Section 6.4)
▪ The required depth to feed distributary 1 at full supply (fig 6.13), case-2	=	6 m	(Section 6.10)
▪ Storage depth for case-1	=	2 meters	
▪ Storage depth for case-2	=	3 meters	

Assume that starting from a no storage situation, the pond is raised slowly. The recommended water surface rise is 0.15 m per hour (Chapter 5 of Canal System Automation Manual).

- Approximate time to achieve storage in case-I = 2/.15 = 13 hours; and
- Approximate time to achieve storage in case-2 = 3/.15 = 20 hours

The oharacteristics of ponding with reference to *the* unsteady state are discussed in Chapter 8.

6.9 AN EXAMPLE OF ROTATION AT 50% HEAD DISCHARGE

Two scenarios of rotation at 50% of discharge in the CRBC are hydraulically evaluated in this section. The purpose is to present the steady state hydraulic behavior of canal structures under rotation. The computation example presented here is not a proposal for implementation, but presents the scope of this type of analysis. The simulation output is presented in Figures 6.12a and 6.12b, and Table 7, for water levels and gate operations. A brief description of the rotation results follows.

The steady state analysis has been carried out for two cycles of operations at 50% of the discharge, with each cycle distributing water to about half of the distributaries. The simulation is performed separately for each cycle, hence the transition from cycle-1 to cycle-2 is not taken into account.

The model is set to compute the gate opening while the design discharge is given as a target. The target is kept the same in both cycles while the selection of the distributaries to open or close is shown in the status column. This Setting is selected to enable easily modifying or changing the rotation,

The canal is divided into reaches, each reach is located between two cross-regulators. For the rotation, the canal is not divided into big groups. Rather, distributaries of one or two reaches are operated simultaneously. The purpose of this selection is to keep the water surface to a certain level throughout the canal to avoid the filling-up and drawdown problems during the transition.

For some of the reaches, computed water levels are higher than the design water levels. For example, in the tail reach of Stage 111, as shown in Figure 6.12b. One of the reasons is that the maximum opening of the head regulator could not be availed (Table 7), because at the lower levels a hydraulic stability is not achieved. This behavior will need to be checked in the field. All of the cross-regulators operate under submerged conditions in both cycles.

The discharge and gate opening of each distributary is given in Table 7. These computations are for the design data using the same discharge coefficient for all gates (0.60). The opening of the distributaries in a reach vary in a big range; the nearest to the cross regulator is half open when compared to the farthest one-which is fully open.

To simulate the actual canal after construction, the model wilt need to be recalibrated.

figure 6.12a: Water levels and regulator operations for Cycles I & II. Rotation at 50% inflow for Stages I & II.

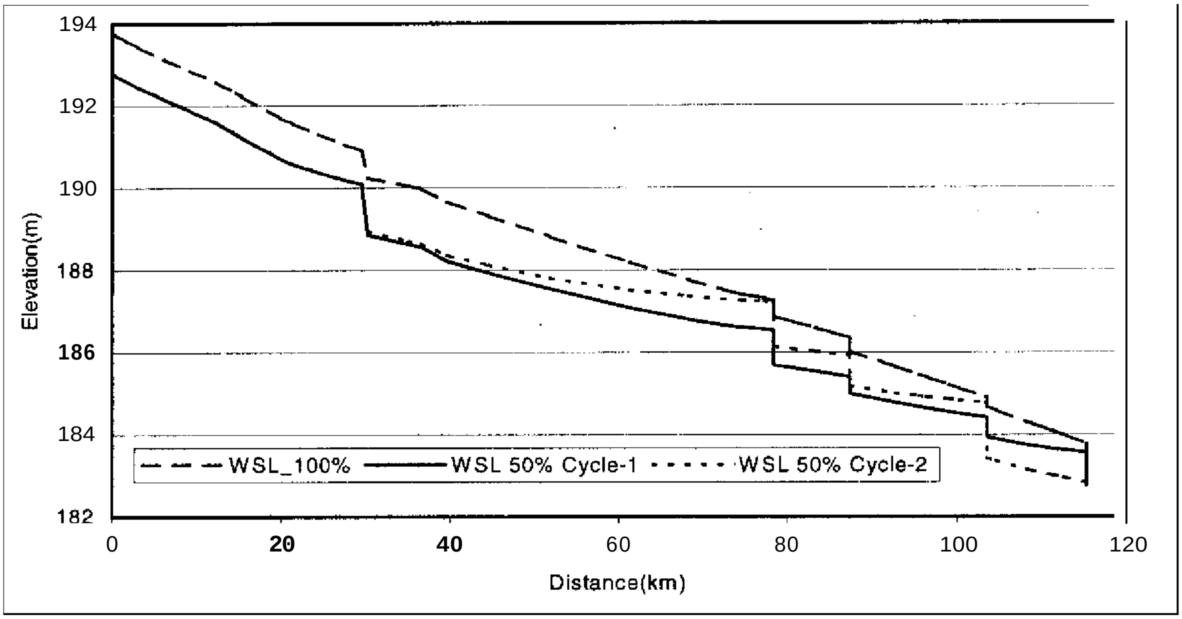


Figure 6.12b: Water levels and regulator operations for Cycles I & II. Rotation at 50% inflow for Stage III.

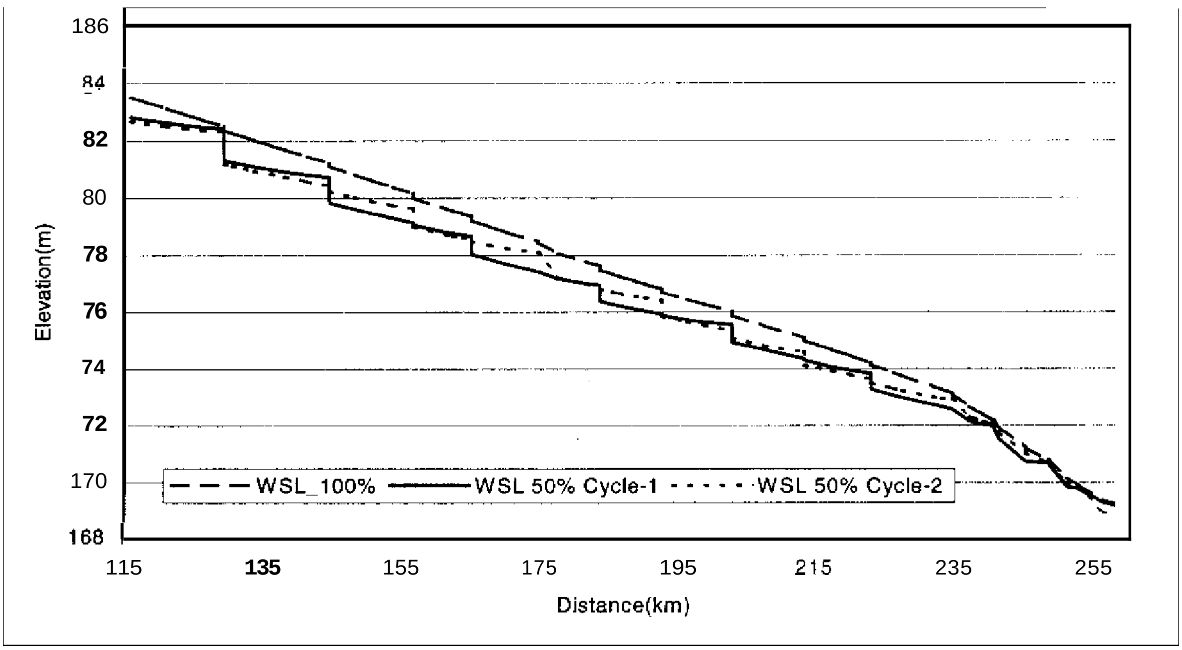


Table 7: Sample output of the hydraulic model for 50% scenario.

SIC STEADY FLOW COMPUTATION RESULTS

REMEMBER: QUALITY OF RESULTS ALWAYS DEPENDS ON QUALITY DATA!

TOTAL DISCHARGE ISSUED AT HEADWORKS	(m^3/s)	67.500
TOTAL TARGETED DISCHARGE AT NODES	(m^3/s)	-60.873
TOTAL SUPPLIED DISCHARGE AT NODES	(m^3/s)	-61.092
TOTAL SEEPAGE OUTFLOW	(m^3/s)	- 3.762
TOTAL OUTFLOW AT TAILENDS	(m^3/s)	2.865

CYCLE I, 1st Week

CROSS DEVICES - HYDRAULIC INFORMATION

CROSS-DEVICE	REGIME	CHAINAGE (m)	DISCHARGE (m^3/s)	U/S LEVEL (m)	D/S LELVEL (m)
S-I RegCombStr	FREE	29498	65.198	190.079	188.907
S-I Trans Weir	SUBM	36575	60.139	188.56	188.432
S-I Reg Tail	SUBM	78344	47.508	186.539	185.696
S-IIReg1	SUBM	87362	47.208	185.407	184.996
S-IIReg2	SUBM	103518	46.05	184.419	183.931
XregS-III	SUBM	115214	39.784	183.551	182.846
S-IIIReg1	SUBM	129410	34.456	182.413	181.32
S-IIIReg2	SUBM	144545	25.463	180.724	179.818
S-IIIReg3	SUBM	156596	25.39	179.127	179.05
S-IIIReg4	SUBM	165052	21.401	178.672	178.023
S-IIIReg5	SUBM	174721	21.343	177.419	177.329
S-IIIReg6	SUBM	183632	16.5	176.948	176.375
S-IIIReg7	SUBM	192538	16.447	175.928	175.884
S-IIIReg8	SUBM	202762	12.871	175.549	174.923
S-IIIReg9	SUBM	213165	12.809	174.356	174.301
S-III Reg-10	SUBM	222791	8.211	173.838	173.264
S-IIIReg11	SUBM	234391	8.142	172.568	172.367
S-IIIReg12	SUBM	240223	5.557	171.985	171.57
S-IIIReg13	SUBM	241168	3.369	171.514	170.964
S-IIIReg14	SUBM	248126	2.93	170.68	170.003
S-IIIReg15	SUBM	252401	2.904	169.784	169.578
S-IIIReg16	SUBM	258684	2.867	169.133	168.983

OFF-TAKES - RESULTS

OFF-TAKE NAME	TARGETED DISCHARGE	STATUS	GATE OPENING (m)
Head	67.5	HEADWORK	
SI_ESC1	0	IMPOSED DISCHARGE ACHIEVED	
TAKKR2	-2.324	GATE - COMPUTED OPENING:	0.667
TAKKR3	-2.324	GATE - COMPUTED OPENING:	0.667
KOTHR2	-3.215	GATE - OFF (Q=0)	
KOTHR3	-3.215	GATE - OFF (Q=0)	
KATHR1	-1.474	GATE - COMPUTED OPENING:	0.397

OFF-TAKE NAME	TARGETED DISCHARGE	STATUS	GATE OPENING (m)
LINKR3	-4.109	GATE - OFF (Q=0)	
D1	-0.714	GATE - OFF (Q=0)	
D2	-1.419	GATE - COMPUTED OPENING:	0.886
D3_1	-3.213	GATE - COMPUTED OPENING:	0.61
D4_1	-5.951	GATE - COMPUTED OPENING:	0.705
SI_ESC2	0	IMPOSED DISCHARGE ACHIEVED	
D5	-3.525	GATE - OFF (Q=0)	
D5A	0	IMPOSED DISCHARGE ACHIEVED	
D6	-2.557	GATE - OFF (Q=0)	
D7A	-0.219	GATE - COMPUTED OPENING:	0.257
D7	-3.224	GATE - OFF (Q=0)	
D7B	-0.085	GATE - COMPUTED OPENING:	0.229
D8A	-0.556	GATE - OFF (Q=0)	
D8B	-0.142	IMPOSED DISCHARGE ACHIEVED	
D8	-0.325	GATE - COMPUTED OPENING:	0.391
D9	-0.461	GATE - COMPUTED OPENING:	0.316
D10	-1.009	GATE - OFF (Q=0)	
D10A	-0.42	GATE - COMPUTED OPENING:	0.362
D11	-0.85	GATE - OFF (Q=0)	
D11A	-2.593	GATE - COMPUTED OPENING:	0.322
D12	-1.751	GATE - OFF (Q=0)	
D13	-3.159	GATE - COMPUTED OPENING:	0.41
SII_ESC1	0	IMPOSED DISCHARGE ACHIEVED	
D14	-4.109	GATE - OFF (Q=0)	
D15	-1.643	GATE - COMPUTED OPENING:	0.825
D15_1	-1.643	GATE - COMPUTED OPENING:	0.825
D16	-1.091	GATE - OFF (Q=0)	
D16_1	-1.091	GATE - OFF (Q=0)	
D16A	-0.453	GATE - COMPUTED OPENING:	0.271
D17	-1.502	GATE - COMPUTED OPENING:	0.429
D18	-2.437	GATE - COMPUTED OPENING:	0.969
D18A	-2.522	GATE - COMPUTED OPENING:	0.893
D19	-2.154	GATE - COMPUTED OPENING:	0.708
D20	-1.587	GATE - COMPUTED OPENING:	0.525
D20A	-1.389	GATE - OFF (Q=0)	
D21	-1.474	GATE - OFF (Q=0)	
D22	-2.012	GATE - OFF (Q=0)	
D23	-1.559	GATE - OFF (Q=0)	
D24	-0.51	GATE - COMPUTED OPENING:	0.472
D24A	-0.822	GATE - COMPUTED OPENING:	0.525
D25	-2.607	GATE - COMPUTED OPENING:	0.884
SIII_ESC1	0	IMPOSED DISCHARGE ACHIEVED	
D26	-0.34	GATE - OFF (Q=0)	
D27	-1.899	GATE - OFF (Q=0)	
D28	-1.672	GATE - OFF (Q=0)	
D29	-2.295	GATE - COMPUTED OPENING:	1.386
D30	-2.494	GATE - COMPUTED OPENING:	0.89
SIII_ESC2	0	IMPOSED DISCHARGE ACHIEVED	

OFF-TAKE NAME	TARGETED DISCHARGE	STATUS	GATE OPENING (m)
D31	-0.623	GATE - OFF (Q=0)	
D32	-1.587	GATE - OFF (Q=0)	
D33	-1.644	GATE - OFF (Q=0)	
D34	-1.474	GATE - COMPUTED OPENING:	0.75
D35	-2.04	GATE - COMPUTED OPENING:	0.762
D36	-1.814	GATE - OFF (Q=0)	
D37	-1.842	GATE - OFF (Q=0)	
D37_1	-1.842	GATE - OFF (Q=0)	
D38	-1.559	GATE - OFF (Q=0)	
D39	-2.267	GATE - COMPUTED OPENING:	1.205
D40	-0.51	GATE - COMPUTED OPENING:	0.464
D41	-1.644	GATE - COMPUTED OPENING:	0.702
D42	-0.708	GATE - OFF (Q=0)	
D43	-1.304	GATE - OFF (Q=0)	
D44	-0.198	GATE - OFF (Q=0)	
SIII_ESC3	0	IMPOSED DISCHARGE ACHIEVED	
D45	-2.267	GATE - COMPUTED OPENING:	1.091
D46	-0.283	GATE - COMPUTED OPENING:	0.257
SIII_ESC4	0	IMPOSED DISCHARGE ACHIEVED	
D47	-2.182	GATE - COMPUTED OPENING:	0.786
D48	-0.397	GATE - COMPUTED OPENING:	0.493
D49	-1.389	GATE - OFF (Q=0)	
D50	-1.644	GATE - OFF (Q=0)	
D51	-2.522	GATE - OFF (Q=0)	
D52	-2.125	GATE - OFF (Q=0)	

CYCLE 2, 2nd Week

CROSS DEVICES - HYDRAULIC INFORMATION

CROSS DEVICE	REGIME	CHAINAGE* (m)	DISCHARGE (m^3/s)	U/S LEVEL (m)	D/S LEVEL (m)
S-IregCombStr	FREE	29498	65.907	190.089	188.987
S-ItransWeir	SUBM	36575	59.066	188.642	188.544
S-IregTail	SUBM	78344	53.55	187.215	186.144
S-IIReg1	SUBM	87362	46.417	185.935	185.173
S-IIReg2	SUBM	103518	41.483	184.775	183.421
XregS-III	SUBM	115214	38.788	182.823	182.71
S-IIIReg1	SUBM	129410	32.344	182.306	181.178
S-IIIReg2	SUBM	144545	32.254	180.419	180.214
S-IIIReg3	SUBM	156596	25.747	179.633	178.997
S-IIIReg4	SUBM	165052	25.697	178.539	178.482
S-IIIReg5	SUBM	174721	21.728	178.089	177.361
S-IIIReg6	SUBM	183632	21.674	176.888	176.803
S-IIIReg7	SUBM	192538	17.482	176.42	175.854
S-IIIReg8	SUBM	202762	15.606	175.329	175.094
S-IIIReg9	SUBM	213165	10.301	174.595	174.129

S-III Reg-10	SUBM	222791	10.243	173.625	173.498
S-IIIReg11	SUBM	234391	8.022	172.914	172.471
S-IIIReg12	SUBM	240223	7.987	172.066	171.835
S-IIIReg13	SUBM	241168	7.981	171.764	171.491
S-IIIReg14	SUBM	248126	6.55	170.676	170.399
S-IIIReg15	SUBM	252401	2.362	169.88	169.35
S-IIIReg16	FREE	258684	0.2	168.924	167.664

KES

OFF-TAKE NAME	TARGETED DISCHARGE	S[illegible]	GATE F[illegible] (m)
Head	67.5	HEADWORK	
SI_ESC1	0	IMPOSED DISCHARGE ACHIEVED	
TAKKR2	-2.324	GATE - OFF (Q=0)	
TAKKR3	-2.324	GATE - OFF (Q=0)	
KOTHR2	-3.215	GATE - COMPUTED OPENING:	0.915
KOTHR3	-3.215	GATE - COMPUTED OPENING:	0.915
KATHR1	-1.474	GATE - OFF (Q=0)	
LINKR3	-4.109	GATE - COMPUTED OPENING:	0.915
D1	-0.714	GATE - COMPUTED OPENING:	0.568
D2	-1.419	GATE - OFF (Q=0)	
D3	-3.213	GATE - OFF (Q=0)	
D4_1	-5.951	GATE - OFF (Q=0)	
SI_ESC2	0	IMPOSED DISCHARGE ACHIEVED	
D5	-3.525	GATE - COMPUTED OPENING:	1.067
D5A	-0.97	IMPOSED DISCHARGE ACHIEVED	
D6	-2.557	GATE - COMPUTED OPENING:	0.276
D7A	-0.219	GATE - OFF (Q=0)	
D7	-3.224	GATE - COMPUTED OPENING:	0.429
D7B	-0.085	GATE - OFF (Q=0)	
D8A	-0.556	GATE - COMPUTED OPENING:	0.423
D8B	0	IMPOSED DISCHARGE ACHIEVED	
D8	-0.325	GATE - OFF (Q=0)	
D9	-0.461	GATE - OFF (Q=0)	
D10	-1.009	GATE - COMPUTED OPENING:	0.411
D10A	-0.42	GATE - OFF (Q=0)	
D11	-0.85	GATE - COMPUTED OPENING:	0.3
D11A	-2.593	GATE - OFF (Q=0)	
D12	-1.751	GATE - COMPUTED OPENING:	0.604
D13	-3.159	GATE - OFF (Q=0)	
SII_ESC1	0	IMPOSED DISCHARGE ACHIEVED	
D14	-4.109	GATE - COMPUTED OPENING:	1.106
D15	-1.643	GATE - OFF (Q=0)	
D15_1	-1.643	GATE - OFF (Q=0)	
D16	-1.091	GATE - COMPUTED OPENING:	0.586
D16_1	-1.091	GATE - COMPUTED OPENING:	0.586
D16A	-0.453	GATE - OFF (Q=0)	
D17	-1.502	GATE - OFF (Q=0)	
D18	-2.437	GATE - OFF (Q=0)	

OFF-TAKE NAME	TARGETED DISCHARGE	STATUS	GATE OPENING (m)
D18A	-2.522	GATE - OFF (Q=0)	
D19	-2.154	GATE - OFF (Q=0)	
D20	-1.587	GATE - OFF (Q=0)	
D20A	-1.389	GATE - COMPUTED OPENING:	0.9
D21	-1.474	GATE - COMPUTED OPENING:	0.793
D22	-2.012	GATE - COMPUTED OPENING:	0.726
D23	-1.559	GATE - COMPUTED OPENING:	0.579
D24	-0.51	GATE - OFF (Q=0)	
D24A	-0.822	GATE - OFF (Q=0)	
D25	-2.607	GATE - OFF (Q=0)	
SIII_ESC1	0	IMPOSED DISCHARGE ACHIEVED	
D26	-0.34	GATE - COMPUTED OPENING:	0.364
D27	-1.899	GATE - COMPUTED OPENING:	0.691
D28	-1.672	GATE - COMPUTED OPENING:	0.542
D29	-2.295	GATE - OFF (Q=0)	
D30	-2.494	GATE - OFF (Q=0)	
SIII_ESC2	0	IMPOSED DISCHARGE ACHIEVED	
D31	-0.623	GATE - COMPUTED OPENING:	0.439
D32	-1.587	GATE - COMPUTED OPENING:	0.584
D33	-1.644	GATE - COMPUTED OPENING:	0.541
D34	-1.474	GATE - OFF (Q=0)	
D35	-2.04	GATE - OFF (Q=0)	
D36	-1.814	GATE - COMPUTED OPENING:	0.78
D37	-1.842	GATE - COMPUTED OPENING:	1.024
D37_1	-1.842	GATE - COMPUTED OPENING:	1.024
D38	-1.559	GATE - COMPUTED OPENING:	0.514
D39	-2.267	GATE - OFF (Q=0)	
D40	-0.51	GATE - OFF (Q=0)	
D41	-1.644	GATE - OFF (Q=0)	
D42	-0.708	GATE - COMPUTED OPENING:	0.887
D43	-1.304	GATE - COMPUTED OPENING:	0.686
D44	-0.198	GATE - OFF (Q=0)	
SIII_ESC3	0	IMPOSED DISCHARGE ACHIEVED	
D45	-2.267	GATE - OFF (Q=0)	
D46	-0.283	GATE - OFF (Q=0)	
SIII_ESC4	0	IMPOSED DISCHARGE ACHIEVED	
D47	-2.182	GATE - OFF (Q=0)	
D48	-0.397	GATE - OFF (Q=0)	
D49	-1.389	GATE - COMPUTED OPENING:	0.525
D50	-1.64	GATE - COMPUTED OPENING:	0.611
D51	-2.522	GATE - COMPUTED OPENING:	0.724
D52	-2.125	GATE - COMPUTED OPENING:	0.929

6.10 THE CURRENT ACTUAL OPERATIONS OF THE SYSTEM

6.10.1 General

The actual operations of the CRBC Stages I & II are briefly described in this section. The information is based upon the field trips carried out during the study, data provided by WAPDA and the data collected by IIMI in 1991-93.

The monitoring reports and IIMI's field measurements (1991-1993) show that off-takes of Stage I draw more water than their allocation during the peak demand period. Proper scheduling procedures have not been adopted yet. Farmers are playing a key role in the operations of the system **by** closing the tertiary outlets when they do not need/want to irrigate their fields. Field observations of eight watercourses show that in December 1991 more than fifty percent of tertiary outlets were used to be closed during the nights. A similar situation is observed in 1998, farmers routinely use less water during night and the gate operators reduce the opening of the head regulators by about 10% to 30% for the night delivery.

The first reaches of the distributaries mostly feed high command areas. The gated structures support a minor canal offtake, while the wooden stop-logs are used **(by** placing them horizontally in the distributary, the grooved side-walls are provided for that) to raise the head for the outlets, The number of wooden logs used by the farmers depends upon the required working head. The width and height of these logs is around one foot, *These structures operate like sharp-crested variable crest weirs.*

The operations of the distributary head regulators are not recorded regularly, white **the** cross-regulators and escapes are controlled and monitored more regularly.

The current operation of the system could be termed as "modified demand-based". WAPOA operators reduce or increase the supply at the head of the distributary when verbally requested by the Irrigation Department's sub-engineer. This request is directly influenced by the farmers' water requirement and water use practices. The written indents are prepared only a few times a year.

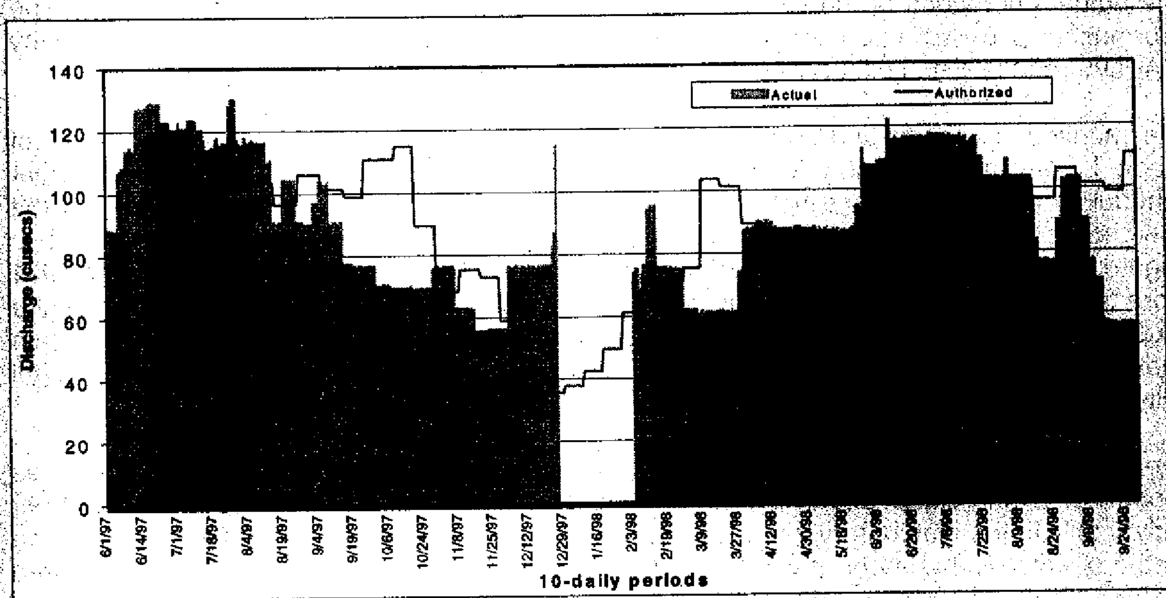
6.10.2 Water Delivery to a Distributary

To estimate the water delivery pattern of a distributary, gauge and gate opening records for eighteen months were collected for Distributary 3 of Stage I, from **WAPDA** D. **I.** Khan. Assuming that these levels have been recorded correctly, the discharge is computed using the calibration done by IIMI in 1993. Figure 6.13 shows the actual discharge hydrograph and the allocation for Distributary 3 according to the WAA. The comparison of both hydrographs shows that:

- The minimum flow is **less** than 1/2 of the maximum **flow** during the year;
- The maximum inflow is slightly higher than the authorized full supply for a few weeks;

- The supply trend shifts from the crop-water requirement hydrograph of PC-1. The influence of the higher water demand for rice is obvious (August, September); and
- The **total** volume delivered during the year is about the same as authorized according to the WAA.

Figure 6.13: The actual and allocated 10-daily discharges for **Distributary 3.**



Chapter

7

7. SEDIMENT DEPOSITION AND TRANSPORT BEHAVIOR

7.1 INTRODUCTION

The sediment deposition phenomenon are among the most problematic ones in the field of irrigation. Although irrigation designers and managers take into account the sediment deposition phenomenon through a regime theory, sedimentation problems still persists. IIMI, in collaboration with French partners, has initiated a program in order to understand first the sediment deposition phenomenon and then to try to provide solutions to the irrigation managers through modeling tools.

The major part of this sediment study is carried out in France. The chosen methodology lies on a statistical analysis of the phenomenon and a modeling with a Global Sedimentary Model (GSM). This model uses the generic approach and consists of simple formalisms to reproduce the sediment behavior of an irrigation canal system subject to sediment deposition problems.

This tool is an independent module that can use the outputs of a hydrodynamic simulation model, Currently, the SIC (Simulation of Irrigation Canals) model of Cemagref France is chosen as the hydrodynamic simulation tool. The full set of hydraulic parameters is used within the GSM and both, graphical and numerical outputs are produced. The GSM is still under development and its current use is limited to the research.

A first calibration of the GSM has been carried out on the Jamrao Canal System in the Sindh Province of Pakistan. The calibrated set of sedimentary laws has been used for the case of the CRBC and the sediment deposition trends have been calculated and are presented.

7.2 MAIN STUDY

7.2.1 Objectives of the Sedimentary Study

The objective of the sedimentary study can be divided into four sub-objectives, i.e.:

1. Representation and understanding of the sedimentary phenomenon;
2. Statistical explanation of the sedimentary phenomenon: development of sedimentary laws;

3. Development of a Global Sedimentary Model (GSM): synthesizing the behavior of the sedimentary laws; and
4. Use of the **GSM** for management applications.

7.2.2 Global Sedimentary Model (GSM)

7.2.2.1 Variables and Necessary Data Set

The primary variables used in the model come from both field observations and hydrodynamic simulations. Three categories of variables can be identified, i.e.:

- Sediment deposition (field data): qualitative (median size d_{50} of the particles being deposited) and quantitative (variation of the canal topography during the studied period);
- Hydraulic (hydrodynamic simulation result): water flow velocity (V), hydraulic gradient (J), hydraulic radius (R_H); and
- Sediment input (field data): average head concentration (C) in particles being deposited and the average head discharge (Q) in the system.

7.2.2.2 Working Set of Variables

Three variables have been chosen from the three categories of variables identified above. See [1] for the explanation of the sedimentary phenomenon in irrigation canal systems.

7.2.2.2.1 Sediment drawing **capacity** (SDC)

The Sediment-Drawing Capacity (SDC) of any off-taking canal, when compared to the parent canal, is studied. Only the content, in particles that may deposit in the system, is dealt with. The definition is the ratio between the sediment concentration in the diversion with the one in the parent canal, i.e.:

$$\boxed{SDC = \frac{C_{d/s\ in\ the\ diversion}}{C_{u/s\ in\ the\ parent\ canal}}}$$

Equation 1: SDC law

7.2.2.2.2 Sediment transport capacity (STC)

The Sediment Transport Capacity (STC) of the water that carries particles of different sizes is studied. Expressed in a generic manner, α, n_1, n_2, n_3 and n_4 are the calibration parameters of the law:

$$\boxed{STC = \alpha . V^{n_1} . R_H^{n_2} . J^{n_3} . d_{50}^{n_4}}$$

Equation 2: STC law

7.2.2.2.3 Sediment deposition (SD)

The Sediment Deposition (SD) law is chosen as the ratio of the deposited weight in a studied reach with the incoming weight of the particles.

$$\boxed{D = \rho_S \,.\, p \,.\, (X_{FIN} - X_{INI}) \,.\, \delta Z \,.\, W}$$

Equation 3: D law

The deposited weight, D, in a studied reach (from X_{INI} and X_{FIN} for instance) is expressed as follows: ρ_S **is** the sediment density, p is the porosity of the deposition, δZ is the variation of the bed level and **W** the width of the canal:

The incoming weight, I, at the head of the irrigation system ***is*** expressed as follows: C is the average head concentration in particles that are deposited, Q is the average head discharge and δt the studied period:

Equation 4: I law

The SD law is then expressed as follows:

$$\boxed{SD = \frac{D}{I}}$$

Equation 5: SD law

7.2.2.3 Application of Sedimentary Laws

Two categories of laws are developed to take into account the sedimentary phenomenon in irrigation canal systems.

7.2.2.3.1 Diversions

The diversions are directly explained **by** the behavior of the SDC law. In the case of **the** CRBC, the SDC law is taken at the constant value of zero. That means each diversion canal is not taking its share of particles **being** deposited. This choice has been carried out because all the crests of the diverting structures of Stages I and II are very high when compared to the main canal bed level, and then, do not draw the particles being deposited, i.e. the coarser ones.

7.2.2.3.2 Linear reaches

The deposition trend in the linear reaches of the main canal is represented through the SD law. **A** calculated SD law (SD_{CALC}) is expressed as a relationship of the STC law, where β and n are the calibration parameters:

$$\boxed{SD_{CALC} = \beta \,.\, STC^{n}}$$

Equation 6: SD_{CALC} law

while an actual SD law (SD_{ACT}) is expressed with Equation 5 according to the field observations.

7.2.3 Management Applications

The three main types of management applications are:

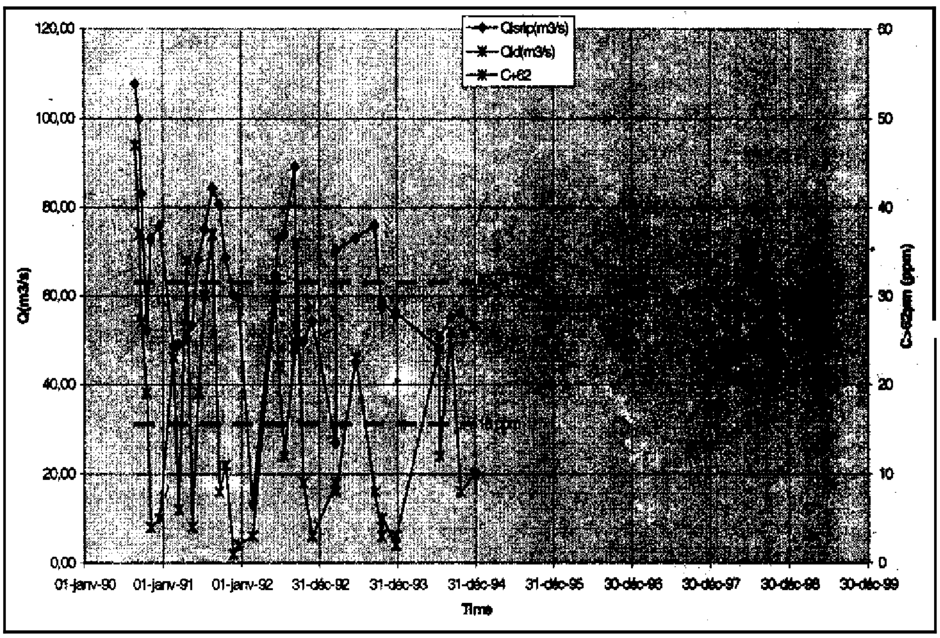
- Diagnosis of irrigation systems subject to sedimentary problems: **inventory** and classification of the problems;
- Operation scenarios to improve the present situation: development and testing; and
- Maintenance scenarios to improve the present situation: **development** and **testing.**

7.3 APPLICATION TO CRBC

7.3.1 Data

Field surveys were conducted by ISRIP[2] between 1990 and 1994. The observations were both, hydraulic and sedimentologic. The discharge, Q, as well as the sediment concentration of particles with a size above 62 microns (particles that may be deposited in the system) **versus** time, are presented on Graph 1.

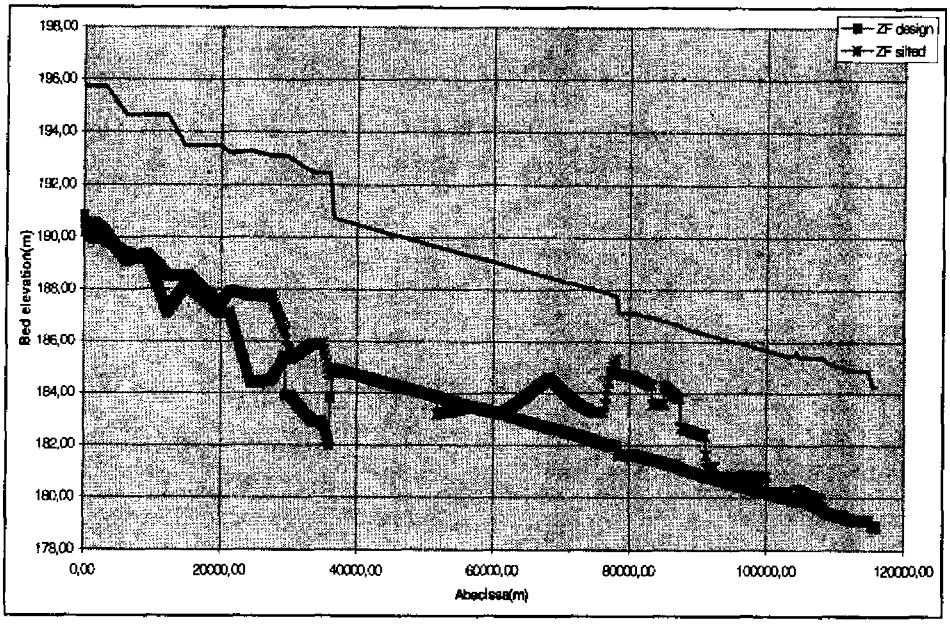
Graph 1: Q and $C_{>62\mu m}$ versus time.



[2] **ISRIP: international Sedimentation Research institute of Pakistan**

Both ISRIP and WAPDA[3] **D. I.** Khan, have collected longitudinal sections of the canal for different times. For the present work, the actual **bed** after the construction (1987) has **been** compared *to* a measured, silted one **(WAPDA,** 1998). The maximum difference in the bed elevation is tremendous, with a maximum value of 3.35 m during about 10 years of sediment deposition. Graph 2 presents longitudinal sections (L-Sections) along the canal abscissa (X).

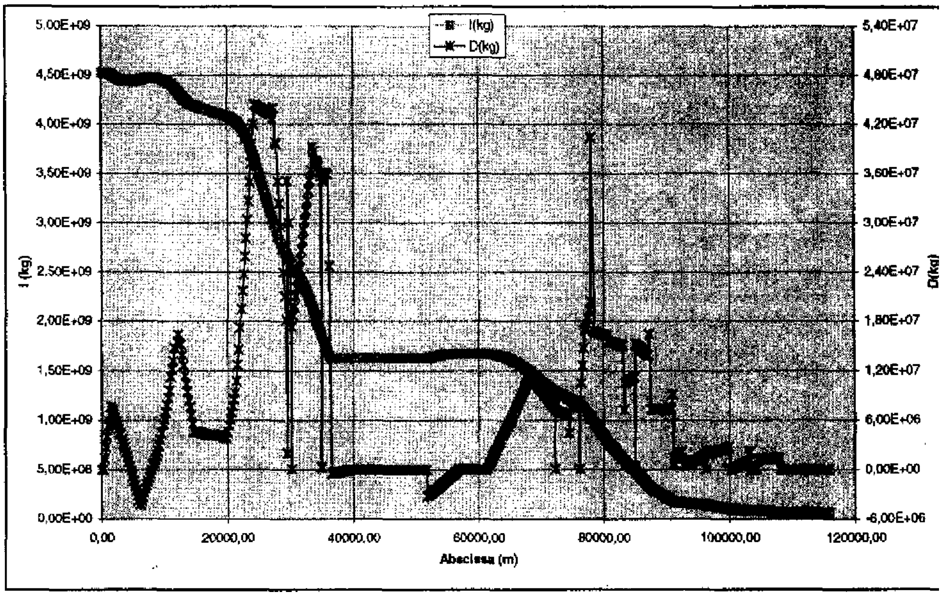
Graph 2: L-sections versus X.



From the variation of **the** canal topography (for both, bed elevation and width), the actual deposited weight along the canal $D_{ACT}(X)$ has been calculated. Then, according to the actual incorning weights at the head of the canal $I_{ACT}(X=0)$, *its* distribution along the canal $I_{ACT}(X)$ is calculated. **A** first value of $I_{ACT}(X=0)$ has been calculated with ISRIP **measurements** of both, the head discharge and sediment concentration (particle size above 62 microns) during **the** considered period. Nevertheless, the obtained value was far **below the** sum of the deposition along the canal, the ratio being in the magnitude of 1/30. From field observations (IIMI Pakistan), the water has been **reported** almost clear of sedimentation at the tail of the canal (tail of Stage II). This boundary condition **has** been used as $I_{ACT}(X=X_{FIN})=0$, and **the** actual incoming weights along the canal, $I_{ACT}(X)$, has been calculated again. Graph 3 presents D_{ACT} as well as I_{ACT} along the canal abscissa (X), taking into account **this** new downstream boundary condition.

[3] **WAPDA: Water and Power Development Authority**

Graph 3: D_{ACT} and I_{ACT} versus X.



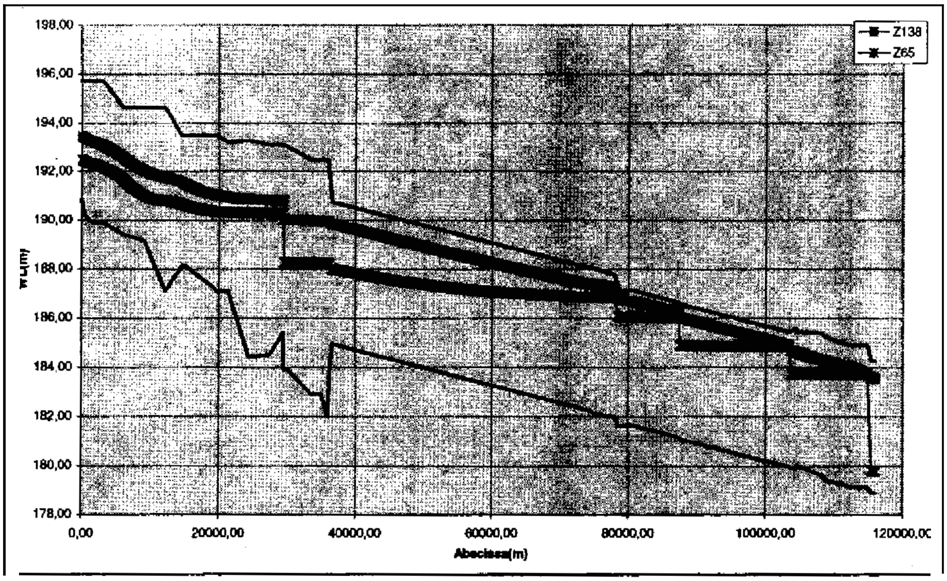
7.3.2 Hydrodynamic Simulations by SIC Model

For the purpose of hydraulic simulations, **the** SIC model has been used, which **solves** the Saint-Vernant fluid mechanics equations for mono-dimensional flow. **Only** the steady state module of the **SIC** has been used.

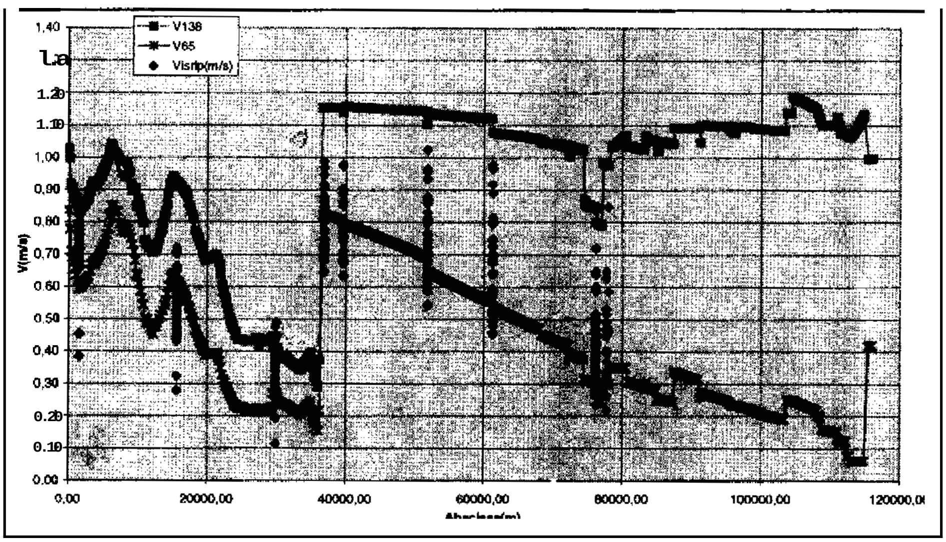
Two operation scenarios have been simulated. **These** correspond with two different head discharges: 4,850 cusecs (138 cumecs) for the design conditions and 2,300 cusecs *(65* cumecs) for the actual flow conditions during the studied period. For both scenarios, all the upstream **water** levels of the regulating structures were **kept** constant **at the design values,** The results are presented on Graphs **4** and 5, respectively, with **the** water **surface** profiles and the water flow velocities along the canal abscissa **(X).** For Graph **5,** the ISRIP hydraulic measurements have been plotted to verify that the calculated velocities are coherent with the field ones.

This set of hydraulic results **is** the input of the **GSM** and its quality is determining the quality of the final output. Indeed, the hydraulic flow conditions rule the sedimentary phenomenon.

Graph 4: Water surface profile versus X.



Graph 5: Water flow velocity versus X.

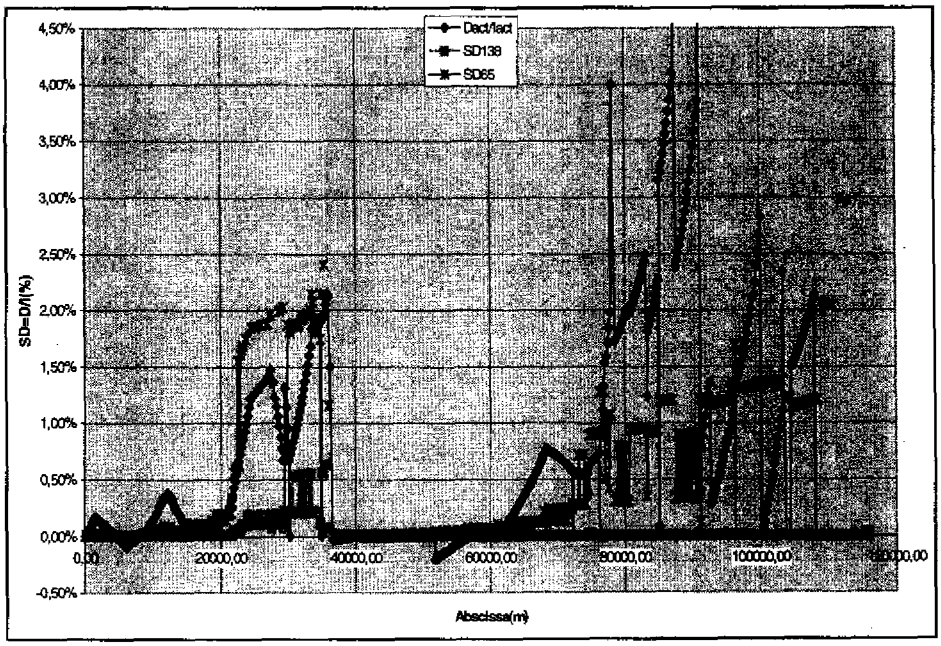


7.3.3 Sedimentary Simulations: GSM Results

A hybrid version of GSM has been used for this work. **Indeed,** the GSM is still under development and the final version *is* not available yet. Nevertheless, the set of sediment deposition laws has been calibrated on a case study of the irrigation canal system (Jamrao) in the Sindh Province of Pakistan; these laws have been applied to the CRBC.

The value of the SD law has been calculated along the canal abscissa (X). The value of $SD_{ACT}=D_{ACT}/I_{ACT}$ is compared to $SD_{CALC\ Q=65}$ and $SD_{CALC\ Q=138}$ along the canal abscissa (X) corresponding to the two operation scenarios.

Graph 6: D/I and SD versus X.

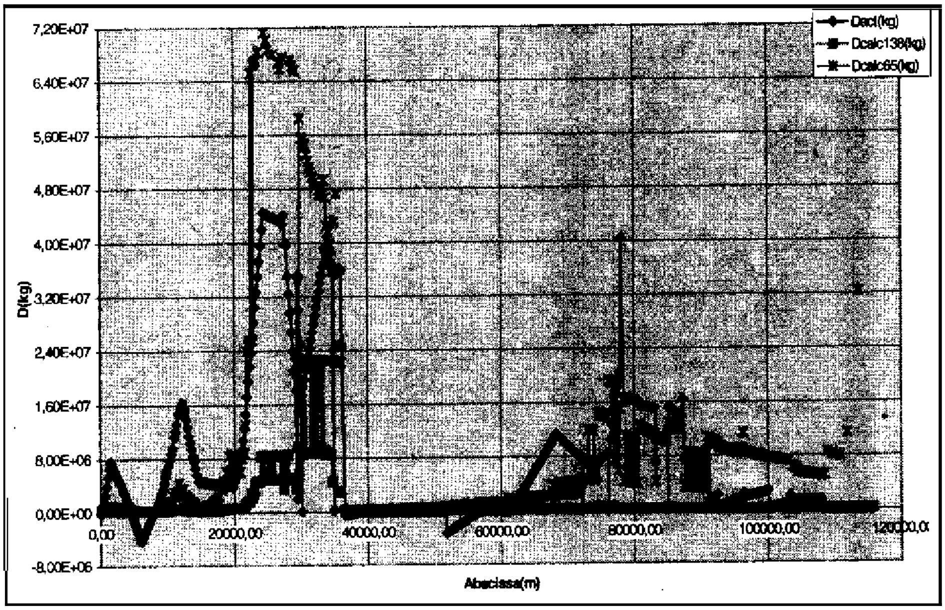


According to the value of $I_{ACT}(X=0)$, the corrected one from the remark of **§0, both** values of D_{CALC} and I_{CALC} have been calculated along the canal abscissa (X). Respectively, Graph 7 and 8 present both, the actual and calculated of D and I, corresponding to the operation scenarios.

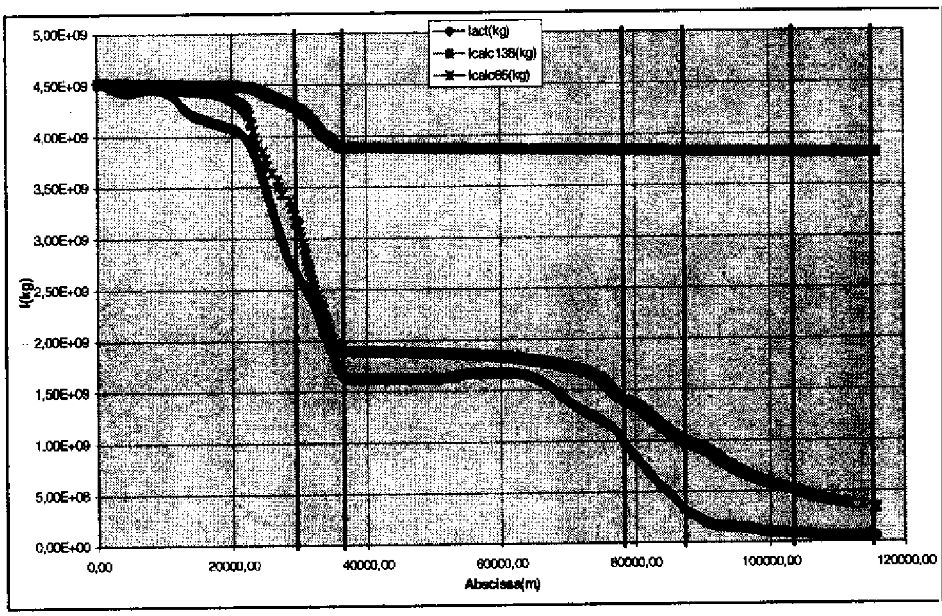
The results are quite encouraging. Indeed, the trends of deposited and incoming weights are well reproduced only **with** the calibrated version of the GSM (see the results for **the actual** and calculated for Q=65 cumecs behaviors). The model **(Q=65 cumecs)** over-estimates the deposition between the 20 to 30 **km,** while it under-estimates that between km 60 to 100 **km. Hence,** the values calculated with the model (Q=65 cumecs) along the canal **are** very similar to the actual ones.

The deposition **trends** under the operation scenario of design conditions (Q=138 cumecs) show that sediment deposition problems should not exist (see Graph 7). But, as the amounts of sediment entering the canal are **the** same, the sediments that **do** not deposit in the first portion are transported towards the tail (120 km), since Stage III is expected to acquire sediment during the coming years. Precautions have to be taken to ensure that sediment deposition problems will not happen in this new portion of the canal.

Graph 7: D versus X.



Graph 8: I versus X.



7.4 CONCLUSION AND PERSPECTIVES

The results of this preliminary sediment analysis are **quite** encouraging. Indeed, the trends **of** sediment deposition are well reproduced **by** the **GSM.** In the CRBC work, only a hybrid version of **GSM** has been used **and** it seems that the coming versions (improved) **may be** a good tool to cope with the sediment deposition problems in this canal.

The problem of sediment deposition is very similar to the one met in the Jamrao Canal System, Sindh Province. The phenomenon, nature of particles and deposition laws match quite well with the CRBC case study.

The development and testing of operation and maintenance scenarios, in order to Improve the present situation, seems an important issue of the work. Indeed, the sediment deposition trend has been affecting the operation and management of the canal.

Chapter

8

8. HYDRAULIC BEHAVIOR UNDER UNSTEADY STATE

8.1 STEADY STATE, THE MOST FAVORED OPERATIONAL CONDITIONS FOR EQUITABLE WATER DISTRIBUTION

An all-time consideration of canal regulation (for upstream **as** well as downstream systems) and operations is to achieve a steady state in the canal and to maintain the best stability of water **levels** and discharge passing through a section. The steady-state conditions are especially important for the gravity channels and most desirable for the duration of a delivery pattern to achieve an equitable distribution and efficient operations *of* the system. However, in actual operations, to maintain a complete steady state is possible only under ideal conditions. Management and physical interventions occurring from the upstream and downstream ends are bound to cause disturbances of different magnitudes and lengths. Logically, there could be three basic reasons for the change of state in a canal or in a canal reach:

a) a change of discharge (inflow), which is scheduled, unscheduled or inherited:

b) a change in delivery pattern; and

c) an emergency operation caused by unexpected operating conditions.

In reality, many long term and short term actions could cause an unsteady **state** in a delivery canal.

- **A** change of allocation from the head
- A shift of rotational schedule
- The operation of a gate to maintain a targeted **level**
- An unscheduled release of storage
- **A** change in delivery pattern
- Farmers' refusal for night irrigation
- Rainfall
- Defective hardware

To minimize the duration of an unsteady state, most of these conditions could **be** improved or managed, but a general target could not be set for the irrigation systems. In fact, the planned delivery schedules, operational and design constraints determine the range and extent to which an unsteady variation could be allowed. For example, there is a difference between supply-based and crop-based systems as well as in the upstream and downstream control systems.

For the CRBC, a uniform discharge is not the target, and the wedge storage is essential to maintain the targeted working head to feed most of the distributaries. These are two major constraints that must be addressed during the planning of an operating scenario.

In the following sections, the unsteady state behavior of different components of the CRBC is analyzed and then a few scenarios for the transition from one steady state to another steady state are discussed.

8.2 CANAL RESPONSIVENESS AND LAG TIMES

8.2.1 Response and Lag Times

The response time of a system is the time required to transit from one steady state to another steady state. The shift from the previous state to a new state is influenced by the definition of both states, the length of the system and the responsiveness of the individual structures. Hence, different sections or reaches of a canal can have different response times. A hydraulic model could be used to integrate the contribution of different influences while moving from one steady state to another. However, the assumptions considered by the model must be taken into account when interpreting the model results.

For the estimation of the response time, two components, lag time and filling time are computed. Before using hydraulic simulation' results to compute the response time (for the design conditions of the CRBC), the theoretical aspects are discussed below.

8.2.2 Travel Time of Unsteady Flow

Wave of Small Height:

The time taken by a sharp wave on a water surface to travel a certain distance, "L", can be approximated by:

$$Tw = \frac{L}{v+c}$$

And

$$c = gy^{.5}$$

Where:

Tw	is the travel time of the wave over the distance L, in s,
L	is the distance, in m,
V	is the velocity in the canal reach, in m/s,

C	is the celerity (wave velocity), in m/s,
G	is the gravitational constant, g = 9.8 m/s^2,
Y	is the water depth in the canal, in m.

In this approximation, diffusion processes are ignored and a constant velocity **is** assumed, which gives a faster time of travel to the flow.

Gradually Varied Monoclinal Wave:

An important percussion regarding operations of the irrigation channel is to gradually introduce an unsteady state, especially when increasing or decreasing a flow in a channel reach. Similarly, a surge should **be** avoided while operating the gates. The uniformly progressive flow in a canal is more equivalent to a monoclinal rising wave. The velocity of this propagation is always higher than the mean velocity of the flow and is computed based on the Manning or Chazy formuta, respectively, as a ratio of **1.67** or 1.50 of the normal velocity.

8.2.3 Standard Equation to Compute Filling-up of a Canal Reach

An approximation formula for the filling time of a canal reach is obtained when the canal reach is simplified as a storage basin, and neglecting the non-uniform flow. This simplification **is** acceptabte when the non-uniformity of the flow can be neglected, for instance, in the case when a regulator at the end of the canal reach determines the water level in the whole canal reach.

The differential equation would **be:**

$$\frac{\delta V}{\delta t} = Q_{in} - Q_{out}$$

Where:

V	is the (storage) volume, in m^3
t	is the time, in **seconds**
Q	is the discharge **'in'** and "out" of the reach, in m^3/sec.

The canal **reach** with a water level control structure at the downstream boundary, transits to an unsteady state when the inflow, "Q_{in}", changes. Consequently, the water level at the control point **will** change, and so, the outflow "Q_{out}".

The time, "Tx%", necessary for a (partial) water level change at the control structure, **can** be approximated with the following formula (see also Figure **8.1).**

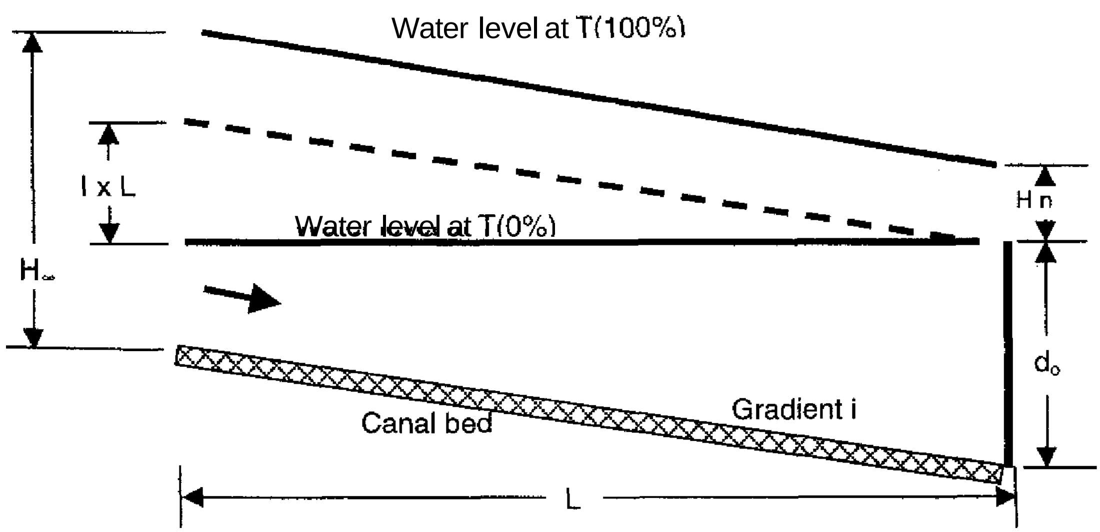
$$T(x\%) = \frac{\frac{1}{2} iL^2(b + 2md_o - \frac{1}{2}\text{miL})}{Q_{in} - Q_{out}} + \frac{HmL(b + md_o + mH\infty)}{ucb_c Hm^u} in\left(\frac{100\%}{100\% - x\%}\right)$$

Where:

T(x%)	**is** the time when x% of the water level change at the control structure ***is*** achieved, in **s,**
L	is the length of the canal reach, in m,

i	**is** the gradient of the canal,
m	**is** the of the side slope, 1V: mH,
b	is the bed width of the canal, in m,
Qin	is the inflowing discharge into the reach at T>0, in m^3/s,
Qout	is the outflowing discharge from the reach at T=0, in m^3/s,
H_o	is the initial ($T_0\%$) energy head above the control, in m,
H_n	is the ultimate ($T_{100\%}$) energy head above the control, **in** m,
Hm	is the **mean** water depth above the control, thus ½Ho + ½Hn, in **m,**
d_o	is the initial water depth above the bed at the control , in **m,**
H	is the equilibrium water depth when Qin = Qout, in m ,
C	is the discharge coefficient of the rating curve at the control,
bc	is width of the control, in m,
u	is the exponent of the rating curve at the control, e.g. u=1.5 for an **overflow** and u=0.5 for an undershot structure.

Figure 8.1 : Filling and response time of a canal reach.



8.2.4 Estimating Lag Times Using Simulation Results

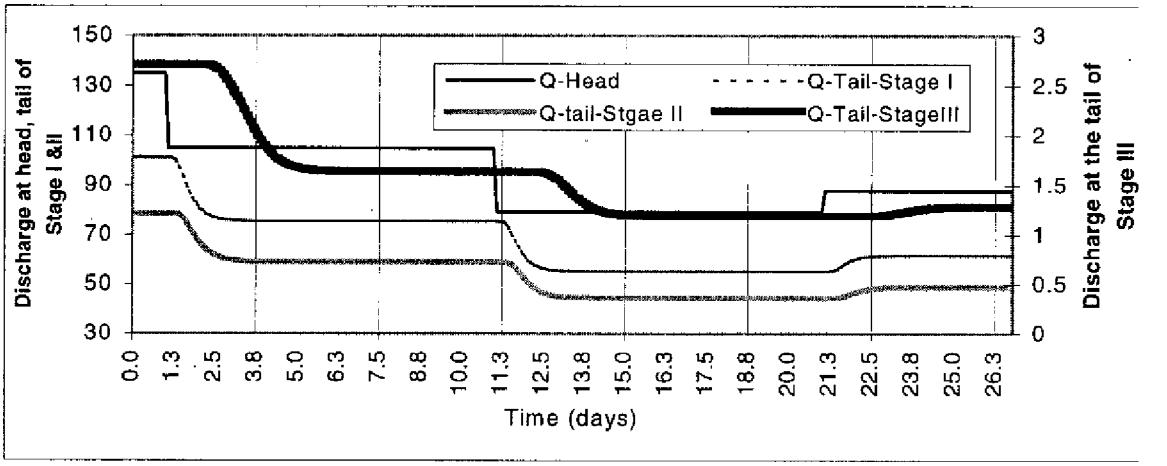
The following steps are adopted **to** estimate the fag times *at* the control structure of the **CRBC.** It must **be** noted **that** the following is an approximate method using the design parameters/assumptions, and the results must be confirmed (through ***a*** validation scenario) from the field before their implementation.

- **the** target steady **states** are defined for 100, 80, 67 and 50% inflow with a proportionate delivery to each off-take;
- simulation is done without any gate operation to have the first approximation of the time lags;
- using these time lags, gates are operated to achieve a proportionate delivery;
- **the** first set of results are cross-checked using spreadsheet computations and improved; and

- the time lags are estimated for the final state when 80% to 100% stability **is** achieved at different structures in a reach.

Figure 8.2 shows flow profiles (discharge line) at the head and tail regulators of each stage of the CRBC for three 10-daily transitions from 100 to 60 percent inflow. It can be seen from the profiles that the lag time for the disturbance to reach from the head to tail is 2 to 3 days, while the time to achieve the next steady state is 3 to **4** days.

Figure 8.2: Flow profiles for four ten-dailies under ideal conditions, at the tail of each stage.



The Figure 8.2 shows that the transition time does not vary in a big range for different discharges. It could be due to two reasons: i) the minimum change of velocity at different discharges because no gate **is** being operated, and ii) when moving from ***a*** higher to lower discharge, filling-up of the reaches is not involved and only transition time is estimated. The rate of change of the water depth is not taken as a constraint at this stage. Hence, it is a case of smooth transition under ideal conditions. The time lags computed for these conditions provide a consistent, but approximate set of lags, which could **be** further improved **by** introducing different constraints.

The behavior of each cross-regulator could be studied in detail for these smooth transitions. As an example, water levels and the response time for five transitions discussed above are shown in Figure 8.3 for **the** cross-regulator at the tail of Stage **I.**

- The time when a disturbance starts reaching a location is smaller for the higher discharges, but the difference of time is nominal.
- The stabilization time is almost double of **the** reaching time of a disturbance.
- The smaller the height of a disturbance, the less the stabilization period (time).
- **A** positive shift is relatively slower than the negation shift.

Based on these computations, two complete scenarios are defined for 30 to 43 and 70 to 100% transitions. The cross-regulators are operated to achieve appropriate working heads for the proportionate delivery to the secondary system.

The estimated time lags at each cross-regulator and weir are plotted in Figure 8.4. Each reach is considered stabilized when delivery to the distributary canal is 80% stabilized. Slightly higher tags (more time) are shown in the first reach of the canal, which is unlined. The net difference of the lag time between two transitions at the tail of the CRBC is 14.5 hours. The lag times in the tail reach of Stage I are relatively smaller due to the availability of storage in that reach. The standard 70% shows the computations without storage, the quantitative influence of storage is obvious.

Figure 8.3: Water levels at the tall of Stage I.

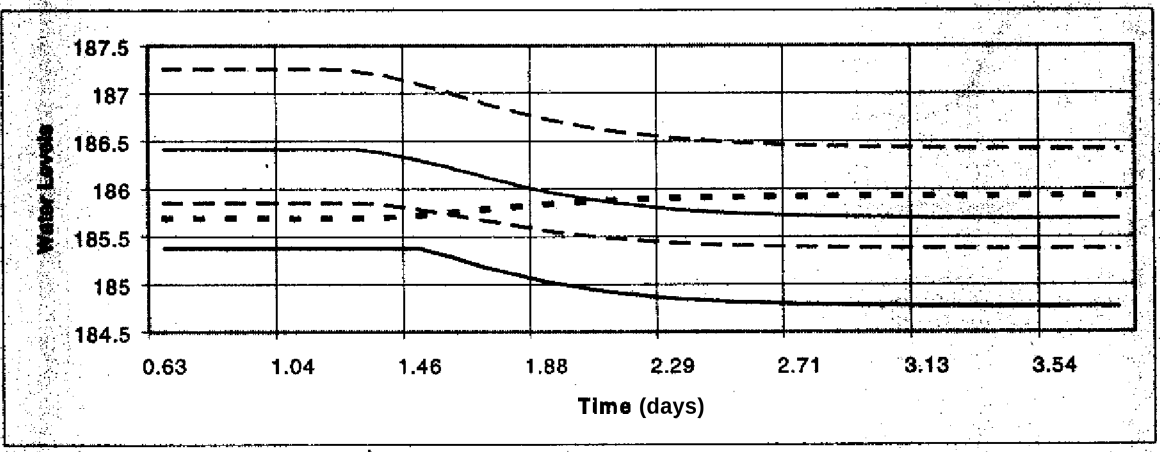
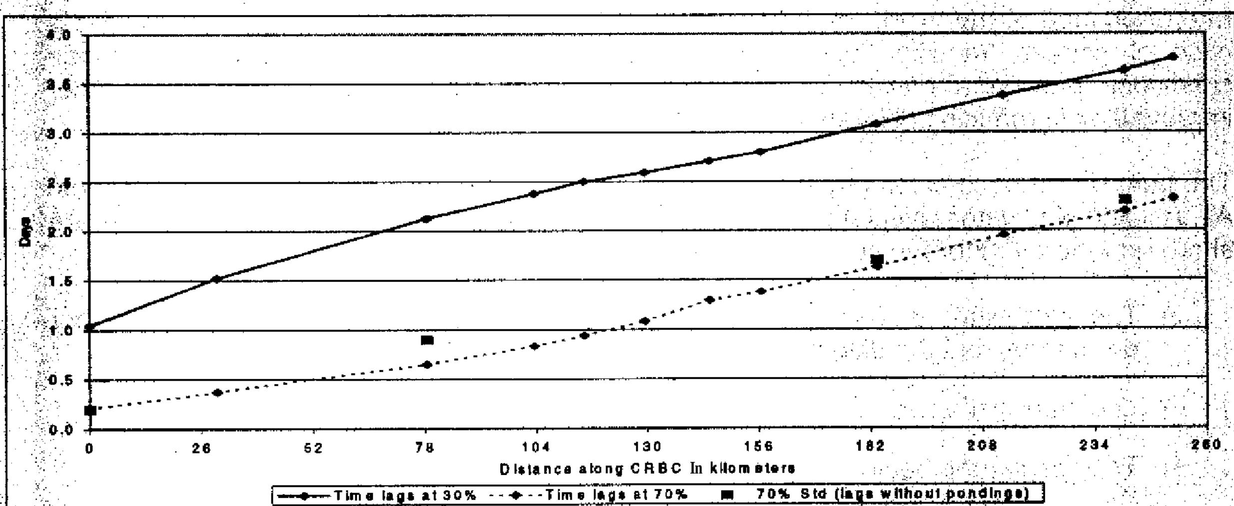


Figure 8.4: Estimated time tags at control structures for 30%-70% and 70%-100% transitions.



8.3 FILLING-UP OF THE CRBC AT 30% INFLOW

It is obvious from the time lag analysis that the filling-up of the reaches is as important as the transition of the flow. The start-up of the conveyance and delivery in the CRBC main canal is simulated to release a supply of 30%. This example presents the process of filling up and the behavior of the canal during the process. The target is *to* achieve a stable delivery in the minimum time period. A description of the scenario is given below.

1. **As** the first step of simulation, the discharge is released from the head and conveyed to the **tail** without **any** operation or delivery. Using the information about conveyance time and the starting water levels in each reach, cross and head regulators are operated to deliver 30% of proportionate flows to each distributary. Preference order is not considered for the distributary operations.

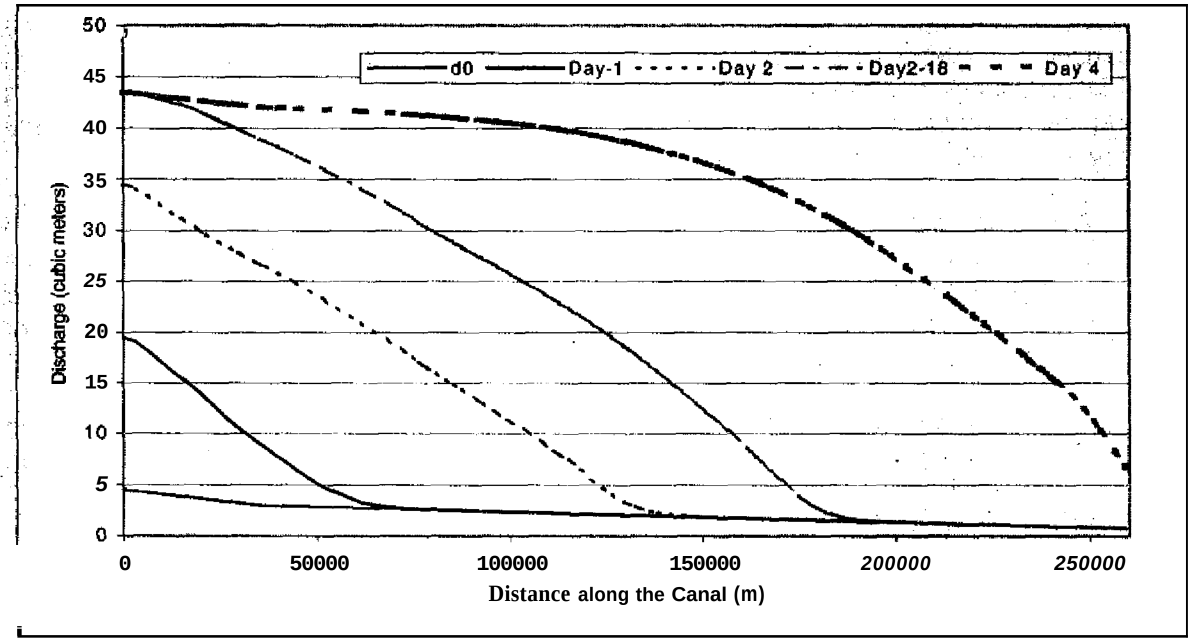
2. **A** base discharge of about ***3*** cubic meters (125 cusecs) is assumed available in the canal at the start of operations. **As** the SIC model **does** not simulate the propagation of a water-front on a dry bed, this discharge provides a thin layer of water on the bed. Most of it, however, **is** consumed **as** seepage losses along the canal.

3. The discharge at the head of the CRBC is released at a rate of 1.5 cumecs per hour (50 cfs/hour) with two four-hourly breaks. The water level at the head of the CRBC increased about I meter in two days. The bigger time gaps can be introduced as is the practice for the operations of big canals, but, the hydraulic model itself does not make a judgment about it. The information provided about water **level** changes could be used to select the desired rate of the water release.

4. The water levels at the lining transition increased at different rates in the lined and unlined sections. The increase is about 0.6 meters in the unlined section, **which** is achieved within two days (about 1/2 day after the last increase at the head) but remained constant afterwards, while, the water levets **in** the lined section are raised **by** 2 meters in two days.

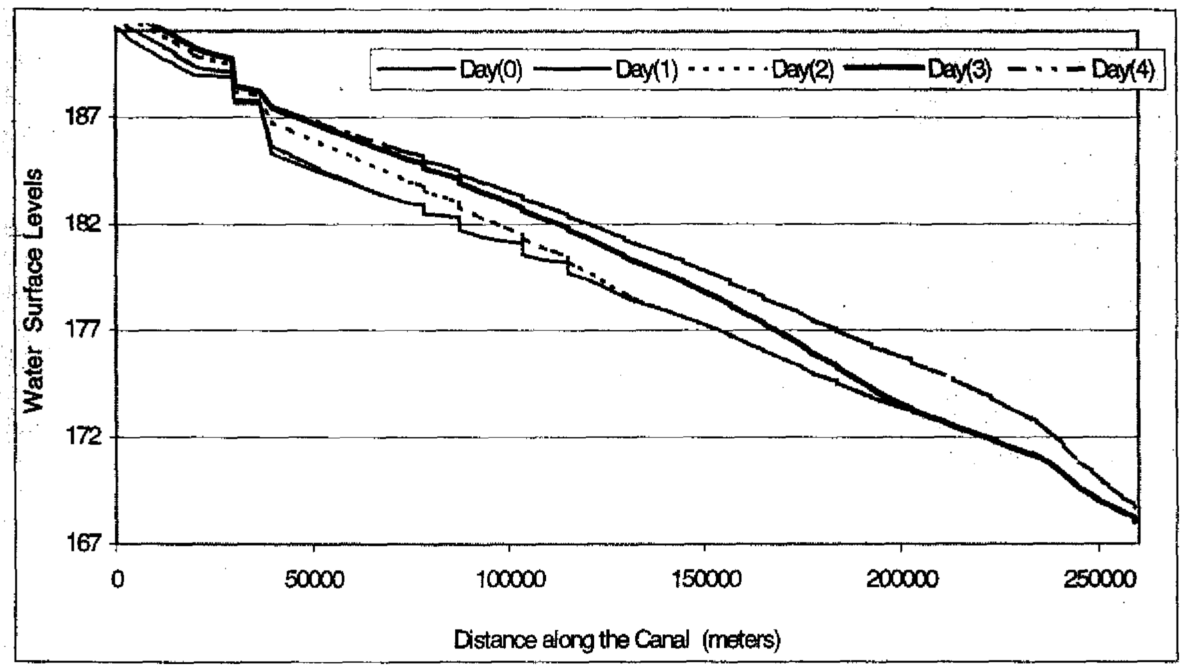
5. The first scenario is carried out without operating any cross or head regulator. The progression of the water-front along the canal is shown in Figure 8.5. The five flow profiles indicate the rate of travel of the water along the canal from day-0 to day-4. **At** the end of day-4, water at **the** tail is higher than the design level. Operation **or** delivery is not made yet. The profile at the end of day-4 also indicates the storage trend of the main canal reaches without gate operations. **The** water levels in the second half of the canal has started rising first, which means that less incremental storage will be required in these reaches.

Figure 8.5: **Discharge progression along the canal at 30% release; no operations.**



The incremental increase of water tevels *for* the same operations is **shown** in **Figure *8.6.***

Figure 8.6: **Progressive water levels at 30% release from the CRBC; no operations.**



6. Based upon the information about progression of the flow'and water-levels, the cross-regulator is operated to achieve the required working level in each reach, and then distributary gates are opened. **For** example, the cross-regulator at the taii of Stage **I** was operated when the **WSL** was increased by 1.5 meters. The level was further raised by 2.5 meters with storage (ponding). The stabilization in this reach is achieved in three days.
7. Figure 8.6 shows progressive water levels, white, the delivery was adjusted to 30%. **A** strict preference order is not followed, especially towards the tail of Stage III**,** because it starts filling **up** before the maximum level **is** attained in the upstream reaches. Heading up **is** not carried out for first two days, then the storage in Stage **I** and II**,** is raised. The progression of flow is much slower than the previous case due to storage and delivery. **A** substantial volume of water is to be stored in canal reaches during the filling-up operations.

a. The tail section *of* Stage III gets about 90% of **its** share in four days, but Stage II still **does** not achieve the maximum levels. Overall, **it takes** five days to achieve a steady state. Some distributaries of Stage II can still not **get** their proportionate share.

9. The pond levels and storage volumes for different reaches can be computed, as shown in Figures 8.8a and 8.8b.

10, The storage depth and filling time are computed by the unsteady state simulations.

Figure 8.7 Progressive water levels for 30% release at the proportionate delivery.

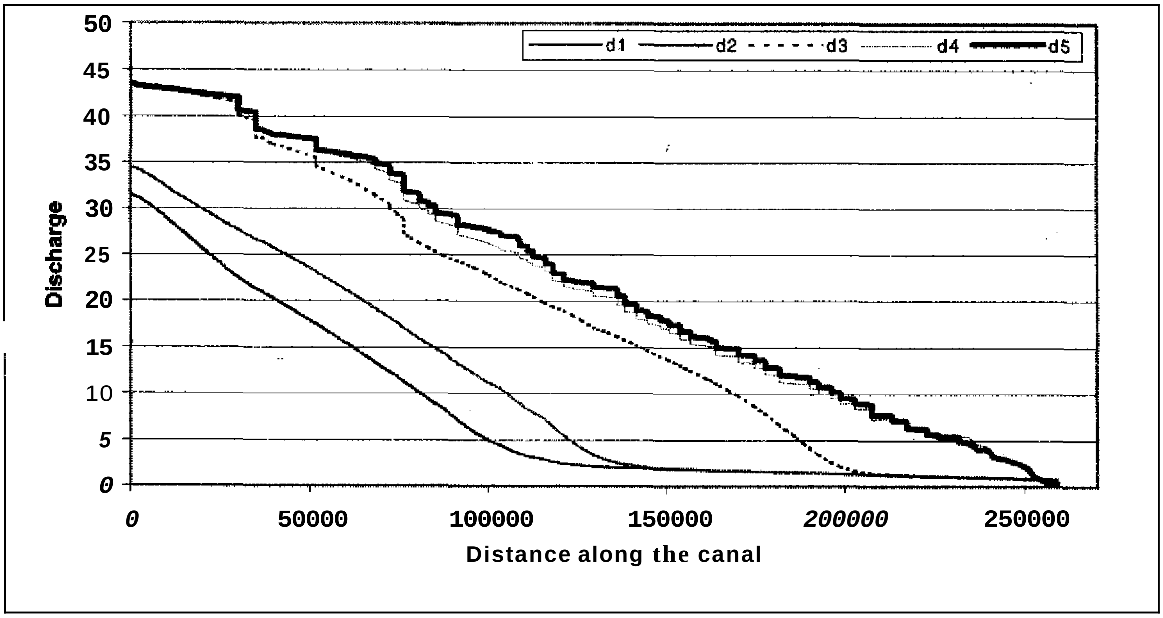
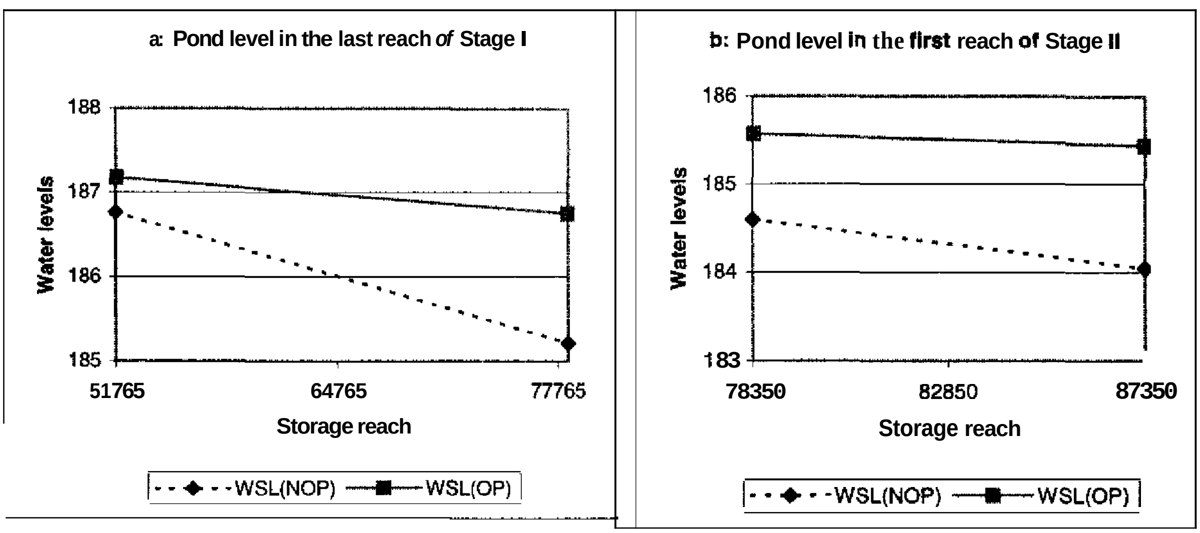


Figure 8.8(ab): The evaluation of the storage depth for 30% proportionate delivery in two reaches.



8.4 SENSITIVITY OF CROSS-REGULATORS

The cross-regulators are the most important control structures in the CRBC for managing water levels at low flows. The well planned water level targets need to be defined and maintained for the efficient performance of a specific schedule, like rotation, or to maintain stability during a transition. Other than main canal influences, operations of the distributaries need to be managed by cross-regulators for any change in the distribution pattern.

All of the cross-regulators of the CRBC operate in submerged conditions at low flows. The farthest distributaries are in the upstream end of the reach and water levels have to be raised all along the reach. As a consequence, influence of a cross-regulator goes much beyond the direct storage reach and affects the operation of upstream and downstream control structures. Therefore, the hydraulic performance or sensitivity of a structure varies, depending upon the requirements of secondary canals and the behavior of upstream and downstream structures and their sensitivity.

The hydraulic behavior of three cross-regulators, in the head, middle and tail reaches of the CRBC is estimated through the simulation analysis. The minimum allocation of the CRBC, 30% discharge, is selected for this scenario because the operation of the cross-regulator will be at a maximum at this flow.

I) Cross-regulator at the tail of Stage II, 115 km (RD 378,000 ft.)

The regulator is operated for half of the day by opening its gates 50% more.

Upstream impact:

The upstream water level drops and the downstream rises as the discharge flowing through the gates increases (Figure 8.9a). The upstream influence goes up to the extent of the Stage I tail regulator, where the downstream water level drops. Discharge passing through the gates increases and eventually, the upstream levels also drop (Figure 8.9b). Figure shows a strong influence with the total drop of the level by 0.7 meters (2 ft.) and 2 cumecs (70 cusecs) less discharge passing for some time. These influences at the tail of Stage I are transmitted to four tail distributaries of Stage I, which are fed from the pond.

Downstream impact:

By opening the regulator, more water is released downstream for half of the day. When the regulator is moved back to its previous position, the water passing down decreases because the storage is to be regained. The influence on the next cross-regulator of Stage III and on a distributary further down is shown in Figures 8.9c and 8.9d. Distributary 18 is located 29 kilometers downstream. The delivery to this distributary is kept constant to access the gate operations required from the operator to maintain a constant supply. The figure shows a continuous increase of the gate opening for one day, until it works as a weir to maintain the delivery.

II) Cross-Regutator 9, Stage III, 213 km (RD 699,359 ft.)

The cross-regulator is operated for half of the day by opening its gates 50% more. This influence on the upstream and downstream regulators and on a distributary in the downstream reach is shown in Figures 8.10a to d.

Upstream impact:

Figure 8.10b shows how the cross-regulator ***8*** **is** affected by the operations of Regulator 9, Stage III. By increasing the gate opening, the pond is released. This influence travels upstream, reducing the pond level of upstream regulators, and goes up to Regulator 1 Stage III. **All these** regulators achieve another stable state after some time, but Regulator 9 is reverted back to its previous position and the storage starts increasing again in all of the reaches between these cross-regulators. The instability continues for two days.

Downstream impact:

The quantitative influence is less strong, as the net storage upstream of Regulator **10 is** relatively **small,** but the instability of water levels continues for a substantial period (more than a day), and influences the distributary operation (Figure 8.10c). **A** distributary of Stage III, D38 at 213 km, needs to be operated at least 10 times during 16 hours (Figure 8.10d).

III) Cross-Regulator 13, Stage III, 241 km (RD 791,233 ft.)

Regulator **13 is** located towards the tail of the CRBC, where storage in the canal reaches is not too strong. To test the sensitivity, the regulator is first closed by 0.2 meters and then opened by 1 meter more. The influence in the upstream and downstream reaches is shown in Figures 8.11 a to d.

As is obvious from the figures, the influence of closing the gates is more pronounced than opening of the gates in both directions. **The** upstream regulator 12 **is** influenced by the increase in water levels, while the downstream regulator **14** is influenced by **a** decrease in the total water diversion. Distributary 47 in the upstream reach gets about 15% more water, while its gate opening is kept constant.

The opening of the cross-regulator does not show a substantial change, which indicates that at these level of operations, the upstream as well as downstream influences are not strong.

Figure 8.9(a-d): The operation of the cross-regulator at the tail of Stage II (115 km).

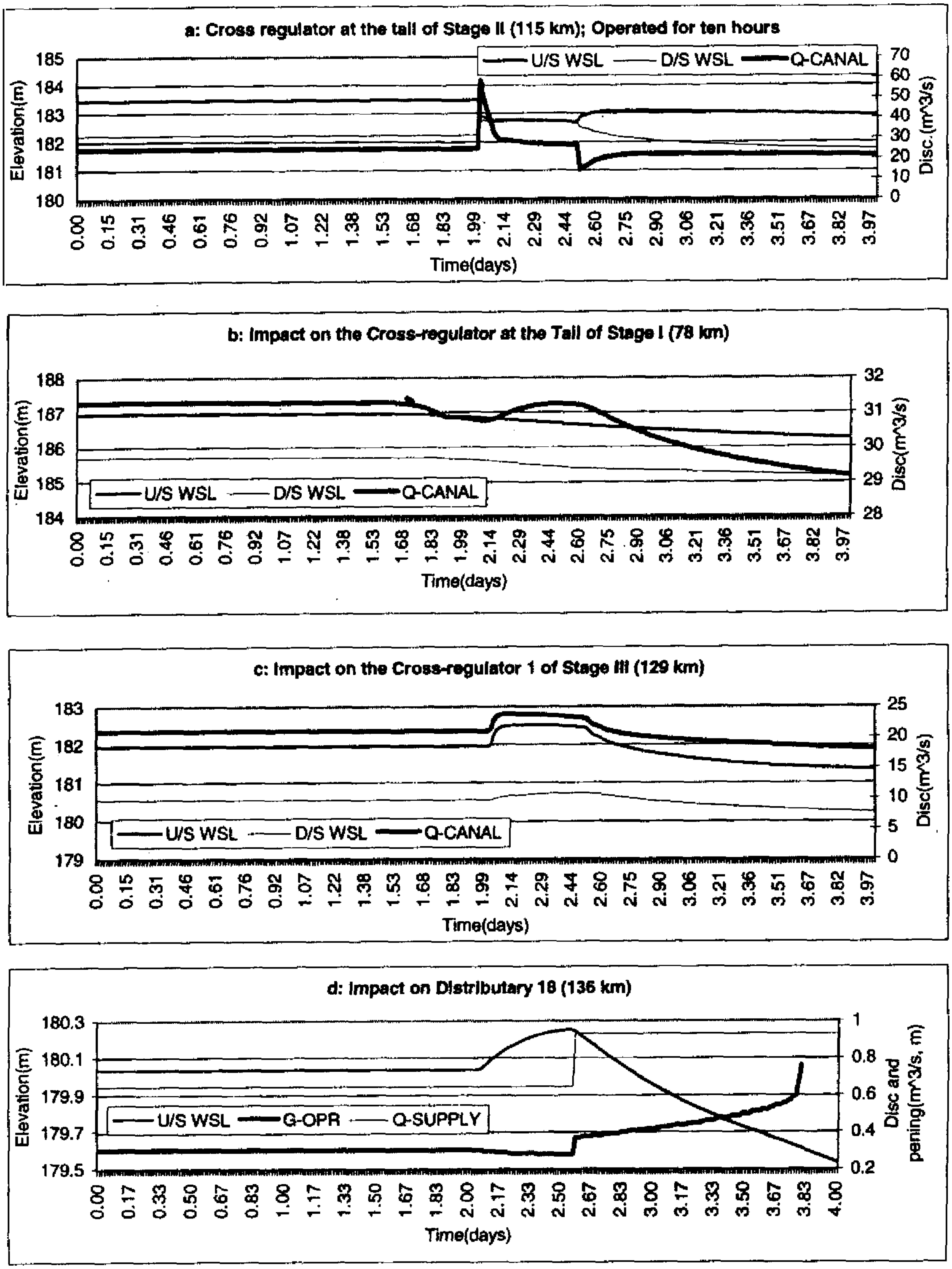
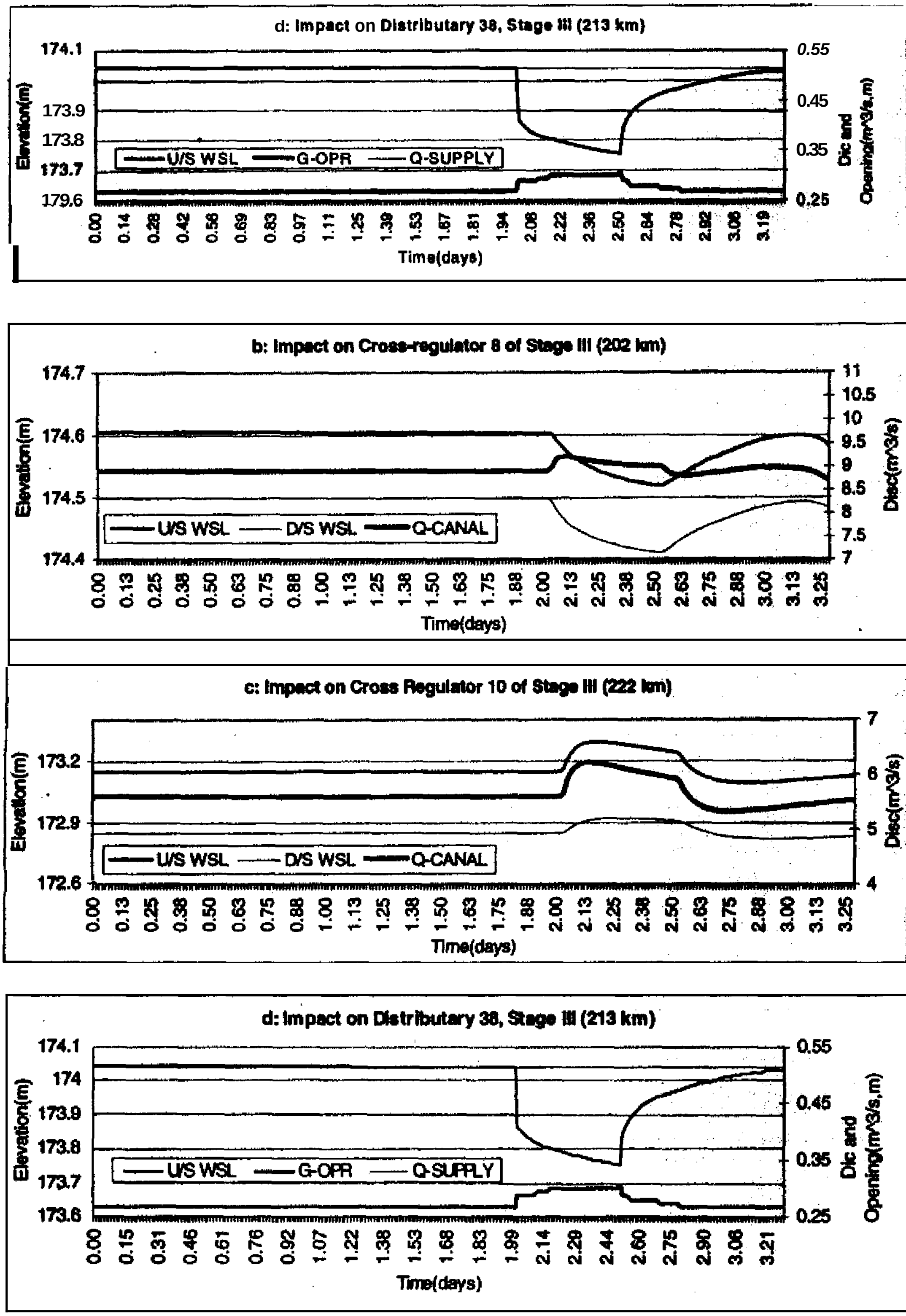


Figure 8.10(a-d): The operation of Cross-regulator 9, Stage III (213 km).

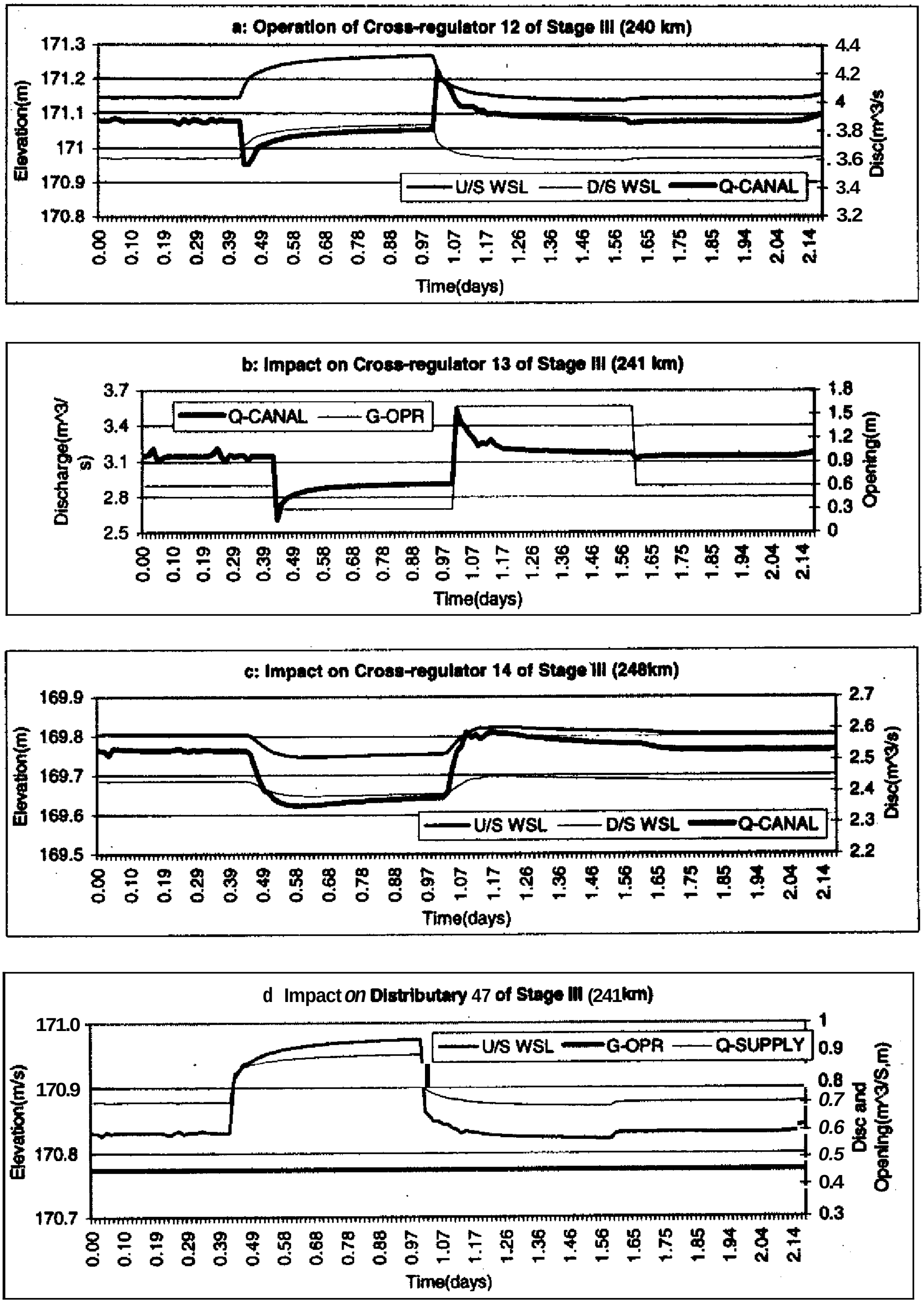
174



The conclusions of this section are:

- The sudden operation of a cross-regulator can create an instability in water levels and water delivery in many upstream and downstream reaches for quite a long period.
- The upstream influence is strong because it affects the storage in all influenced reaches.
- The cross-regulators in Stages I and II are more sensitive.
- To maintain an equitable delivery will be difficult because the number of gate operations to be carried out as a response of an unplanned operation are numerous.
- The discharge passing through a cross-regulator is very sensitive to a change in the working head caused by upstream or/and downstream water levels.
- A positive or negative wave (in terms of the level and discharges) will travel downstream due to an increase or decrease in the water quantity passing through a regulator. A quantitative imbalance in different reaches may generate unexpected responses from different operators who will try to maintain their delivery.

Figure 8.11(a-d): The operation of Cross-regulator 12, Stage III (240 km).



8.5 THE STORAGE LEVEL AND ITS SENSITIVITY

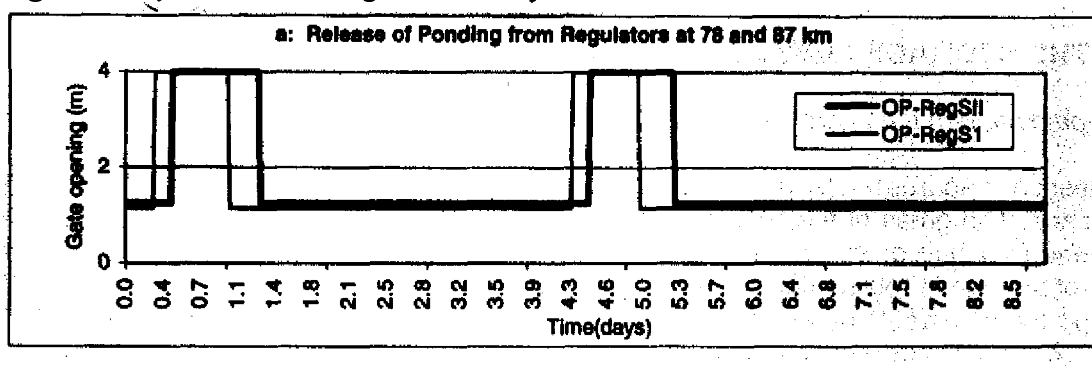
What would happen if a certain amount of storage is suddenly released from a canal reach?

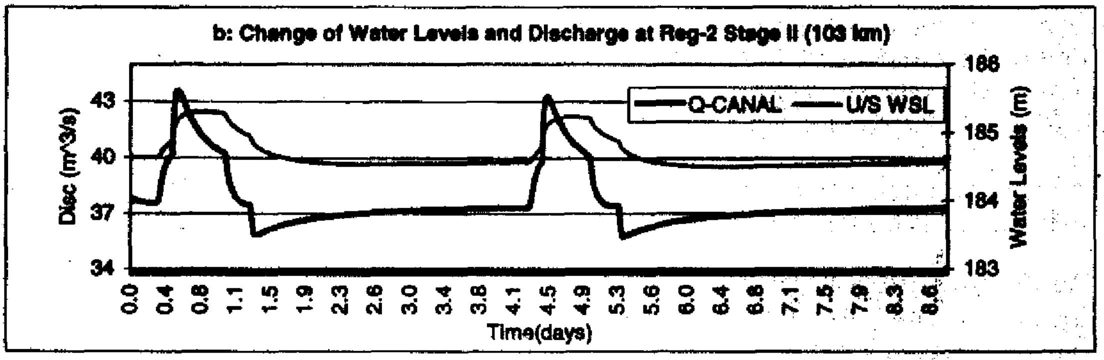
This question is addressed by releasing the storage for a fixed time period and by simulating its influence. The gates of the tail regulator of Stage I and the first regulator of Stage II are fully opened for half of the day and then brought to the previous positions. A reasonable amount of storage is released from the Stage I regulator, which is allowed to travel down by the Stage II cross-regulator with a small enhancement. The operation is repeated after four days. No escape is operated during the simulation.

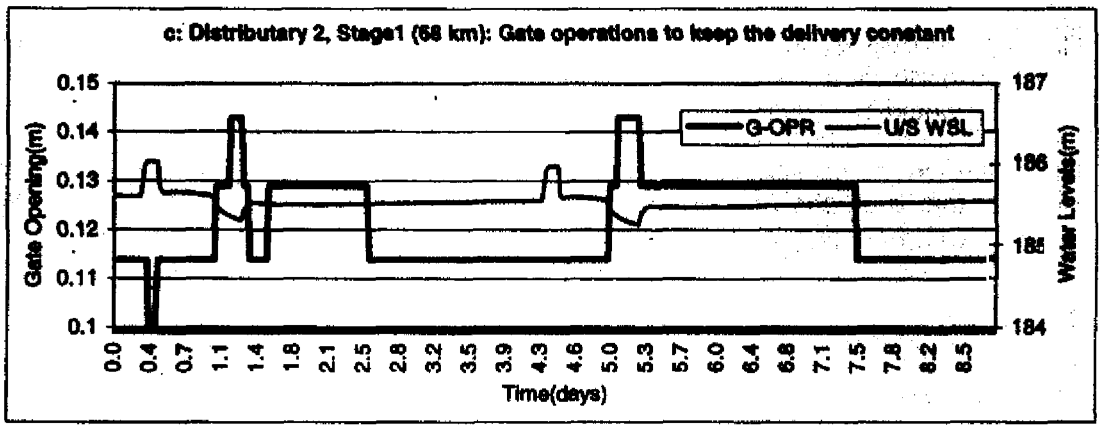
To keep a constant delivery to the secondary system, the discharge is fixed for each distributary and the model is allowed to adjust the gate operation to achieve this target. In this way, an extra release is not shared by the distributaries. The response, as shown in Figure 8.12a to 8.12d, can be summarized as:

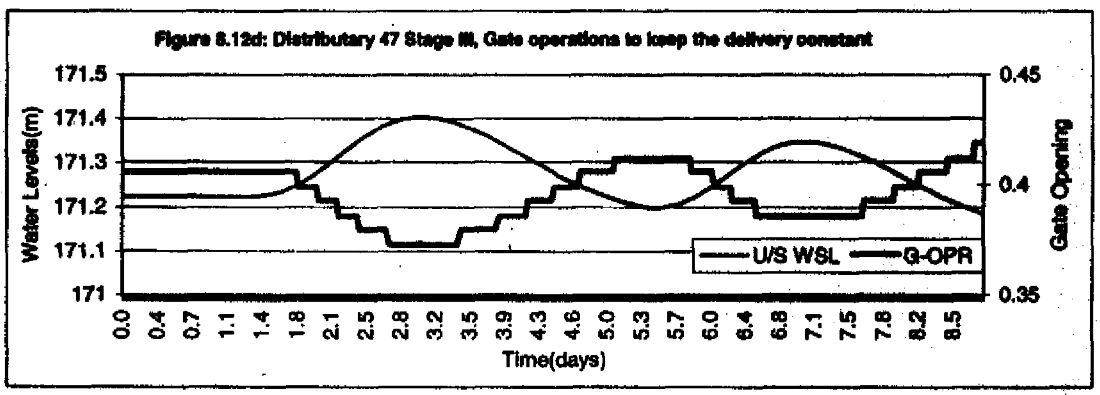
- A capacity problem is not faced in the downstream reaches because the total volume in the canal is less than its maximum capacity and could easily be managed.
- As the water front flows downstream, its height is reduced but the stability time is increased. The maximum discharge at the tail regulator is about 60% extra, while it stays unstable for more than eight days (accumulated impact of all direct and indirect responses).
- To keep the delivery to the secondary system constant, a number of gate opening would be performed, as shown in Figure 8.12c and 8.12d. To manage this type of instability would be a difficult manual task.
- The influence of the volume change is enhanced by the level change, especially towards the downstream, where, the primary responses are followed by the secondary responses and instability prevails for a longer time period.

Figure 8.12(a-d: A storage sensitivity scenario.









8.6 THE ROLE OF AN ESCAPE FOR WATER MANAGEMENT WITHIN THE CRBC

The escape channels for the CRBC are as important as the cross-regulators. The role of cross-regulators is to manage the water levels at low flows within the system. The escapes are required to manage excess water to keep a water balance by diverting **a** part of it out of the system. The seven big escape channels are provided in the CRBC main canal to perform the following functions:

1. To divert extra water back to the Indus River through surface drains when **the** supply is higher than the demand.
2. **To** provide safety in case of an emergency or rains, by diverting water out of the main canal.
3. To maintain a water balance in the system when the extra operational supplies are available in a certain reach.
4. To flush the sediment load out of the system.

To perform these functions, the relevant features of an escape should be:

- a water and sediment-carrying capacity;
- a proper range of influence;
- a required level of influence; and
- a reasonable speed of influence.

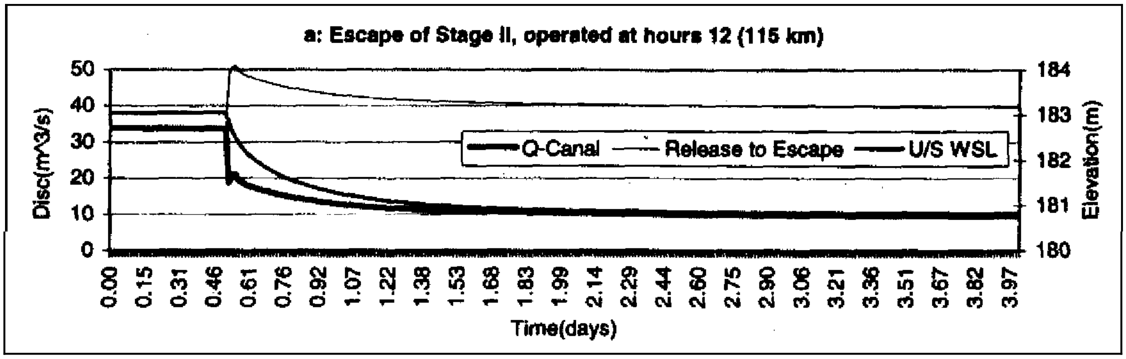
The hydraulic influence of the escape at the tail of Stage II *is* simulated, which is the most important escape due to its location and capacity of the canal in upstream and downstream sections. The simulation results are shown in Figures 8.13a to 8.13h.

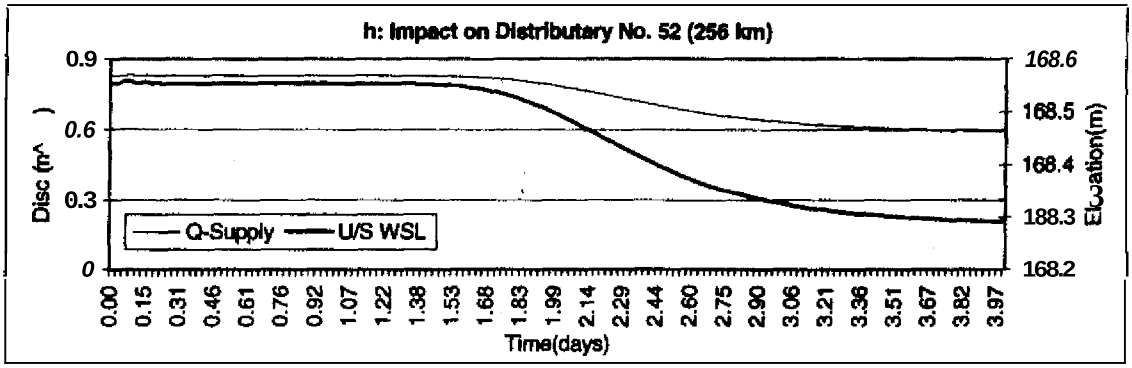
- About 1 m^3/sec. 35 cusecs/sec. of discharge is diverted to the escape for four hours. The water levels drops very quickly, at a rate of 0.35 meters an hour, which is much higher than the recommended drop (0.15 m/sec) of levels.
- The influence in one upstream reach and in two downstream reaches is the maximum, water levels drop lower than the crests of distributaries in these reaches and their water **supply stops.**
- The upstream reaches are influenced by the change of storage and an increase in the velocity; this influence increases to 30 km.
- **A** prominent influence travels to the location of upstream and downstream escapes, hence, the escape is effective in its expected range of operation.
- Towards the tail, the influence of the escape is minimum if another escape **is** not operated.
- The influence starts reaching the tail after a day, and is maximum after 2.5 days.

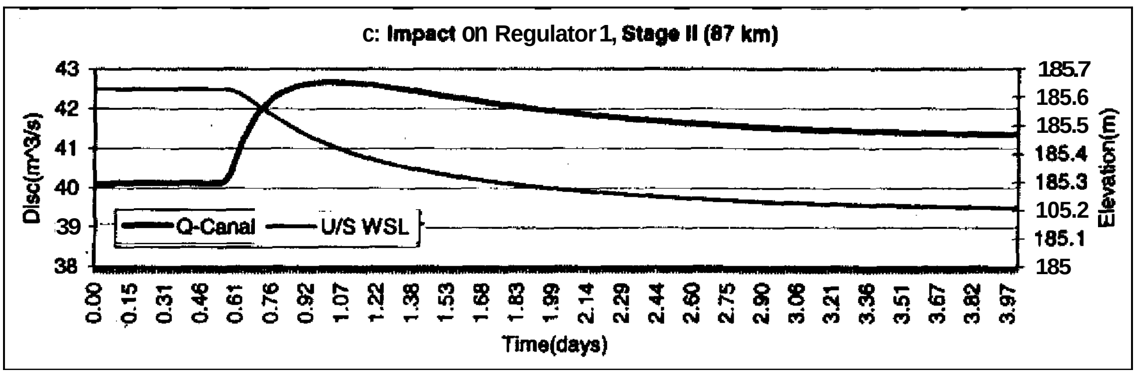
- The influence on a few distributaries is shown in Figure 8.13.
- The unplanned operation of an escape can seriously affect the water delivery to a distributary.
- An escape must be available for the operations if defined in the operation plans.

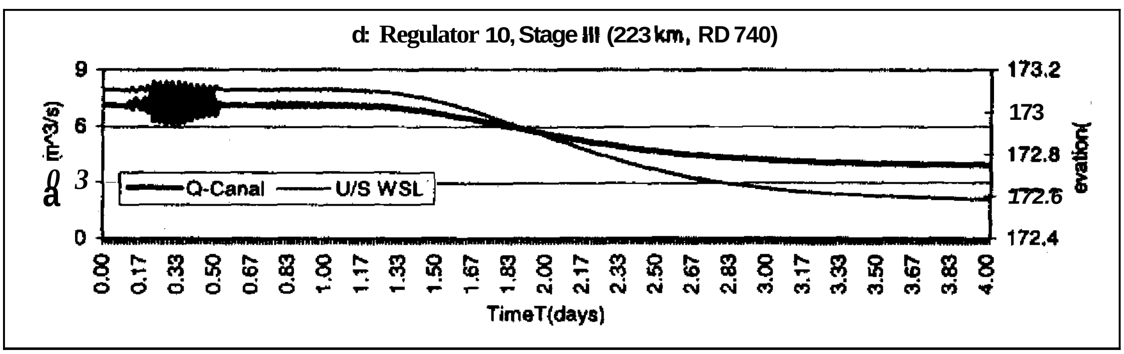
Conclusion: The escapes, like cross-regulators, are the essential but very sensitive control structures, need to be used carefully for the water management in CRBC in case of .emergency or transition.

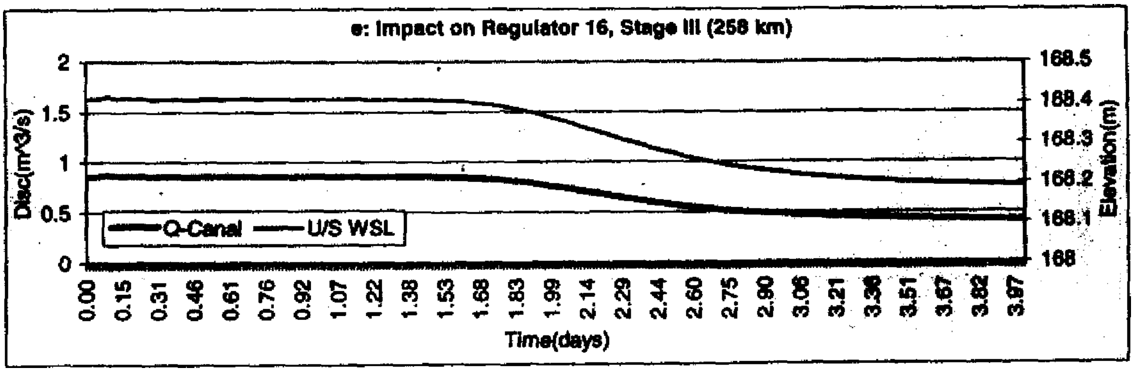
Figure 8.13(a-h): The operation of an escape and *the* range of its influence.

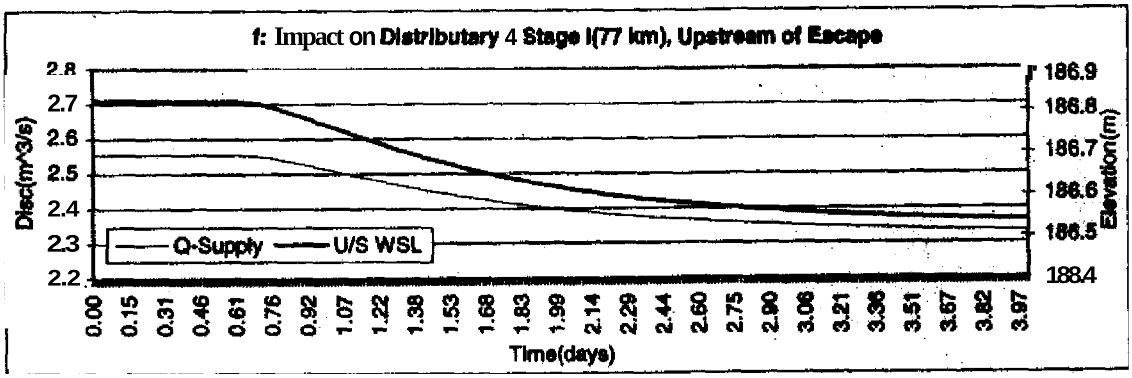


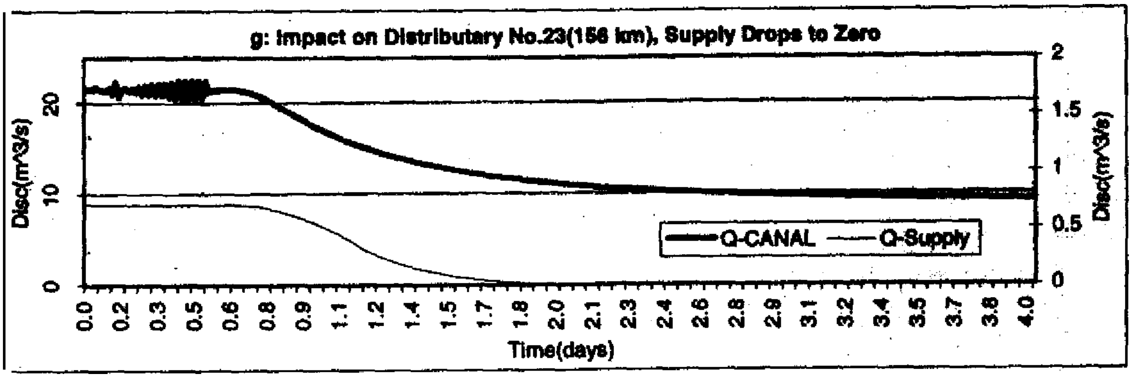


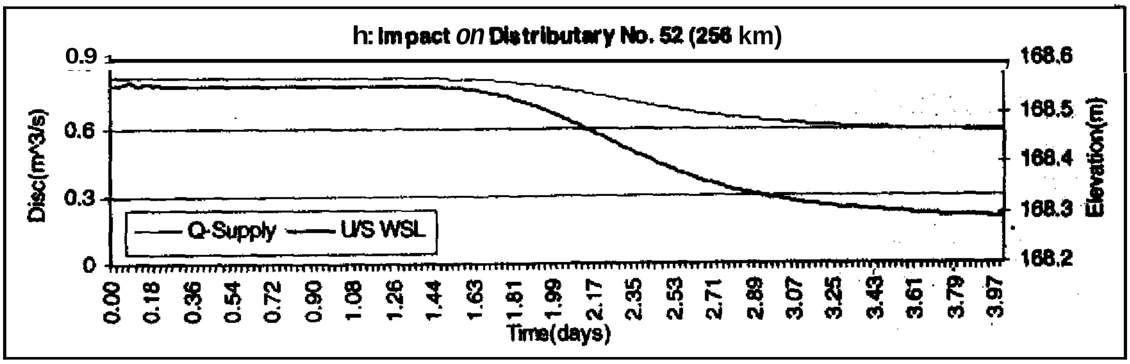





e: Impact on Regulator 16, Stage III (258 km)
Disc(m^3/s)
Elevation(m)
Q-Canal
U/S WSL
Time(days)


f: Impact on Distributary 4 Stage I(77 km), Upstream of Escape
Disc(m^3/s)
Elevation(m)
Q-Supply
U/S WSL
Time(days)


g: Impact on Distributary No.23(156 km), Supply Drops to Zero
Disc(m^3/s)
Disc(m^3/s)
Q-CANAL
Q-Supply
Time(days)


h: Impact on Distributary No. 52 (256 km)
Disc(m^3/s)
Elevation(m)
Q-Supply
U/S WSL
Time(days)

8.7 THE FLOW TRANSITION IN THE CRBC

The Analysis of the canal functioning under steady and unsteady state conditions carried out in the previous sections provides sufficient knowledge about sensitivity range of the canal reaches and structures. Using this knowledge, three sets of complete transition from one inflow-delivery pattern to another are simulated in this section. Two of the transitions are at low flows, while one is at a high flow. This covers the full supply range of the CRBC, as per the ten-daily allocations of the Water Apportionment Accord (see Chapter 3).

The head supply changes (as a percent of maximum authorization) during transition from:

- 30% to 43%
- 43% to 70%
- 67% to 100%

A proportionate supply and distribution pattern is adopted to acquire comparable water levels and other output parameters at the canal reaches and off-takes. The threshold set-up of the operations provided by these scenarios could easily be extended for a different combination of operations, reach to reach or structure to structure.

The following information from the previous chapters are used for the development of these scenarios.

- Water levels and gate openings computed by steady state simulation
- Time lag estimations
- Response time or flow propagation information
- The storage capacity, volume and levels for each flow rate
- Sensitivity of cross-regulators and escapes

8.7.1. Transition From 30% to 43% Inflow

A discharge of 41.5 cubic meters or 1,464 (30%) cubic feet per second is the lowest allocation for the CRBC according to the Water Allocation Accord, based upon crop water requirements of the proposed cropping patterns. The recommendation by design consultants of Stage III has been to increase this level to 43%.

It is assumped that the canal is operated at both flow rates, 30% and 43%, and a transition occur from the first state to the second state. A complete simulation of both situations and the transition is carried out to estimate the depth of fluctuations, transition time and the required operations of different structures.

The immediate operating target of a manager would be to achieve a steady state in the canal as early as possible by minimizing the level of fluctuation at each structure. His ultimate supply target is to maintain an equitable distribution of water by giving a precise time-discharge chart. The following analysis indicates the types of compromises that would be imposed between these targets by the hydraulics of the canal.

The transition is set like this:

- Starting from **a** steady state, proportionately operating at 30% of the discharge, the head release and distribution is increased to 43% (i.e. 58 cubic meters).
- The water levels in each reach are maintained by operating the cross-regulators to achieve 43% delivery.
- Head regulators are operated to achieve a new delivery pattern using the time lags and gate opening computed in the previous sections.
- The operations required to minimize fluctuations are compared with the operations required to maximize equity **by** simulating and comparing two operating methods, i.e.:

  a) changing the head regulator opening to the target in a single step, and

  b) increasing the distributary discharge to the target level in a single step.

The shift in discharge from 30% to **43%** is not too big and the single-step operations at each structure are carried out for an appropriate comparison. **The** field operators may decide to achieve the final opening or discharge in more than one step.

- The discharge at the head is raised at a rate of 1.5 (53 cusecs) cubic meters per hour.

The results of analysis are presented in Figures 8.14 for **the** selected locations. The main indications of the results are:

1) The water depth increases freely in the first 30 kilometers of the CRBC. The remaining canal reaches already have a maximum storage, hence, the water levels do not change much in these reaches. To compare the behavior of water levels in lined and unlined sections, the rate of change of the water depth on both sides of the lining transition is shown in Figure 8.14a. The net increase in the water level on the unlined side is about 0.3 meters (1 ft), achieved in 12 hours. Towards the lined side, depth increases rapidly in the beginning, but gradually slows down. In the first 7 hours, levels are raised by 0.25 meters, while in the next 12 hours, only 0.15 meter increase is computed. The net increase of 0.6 meters is achieved in more than a day.

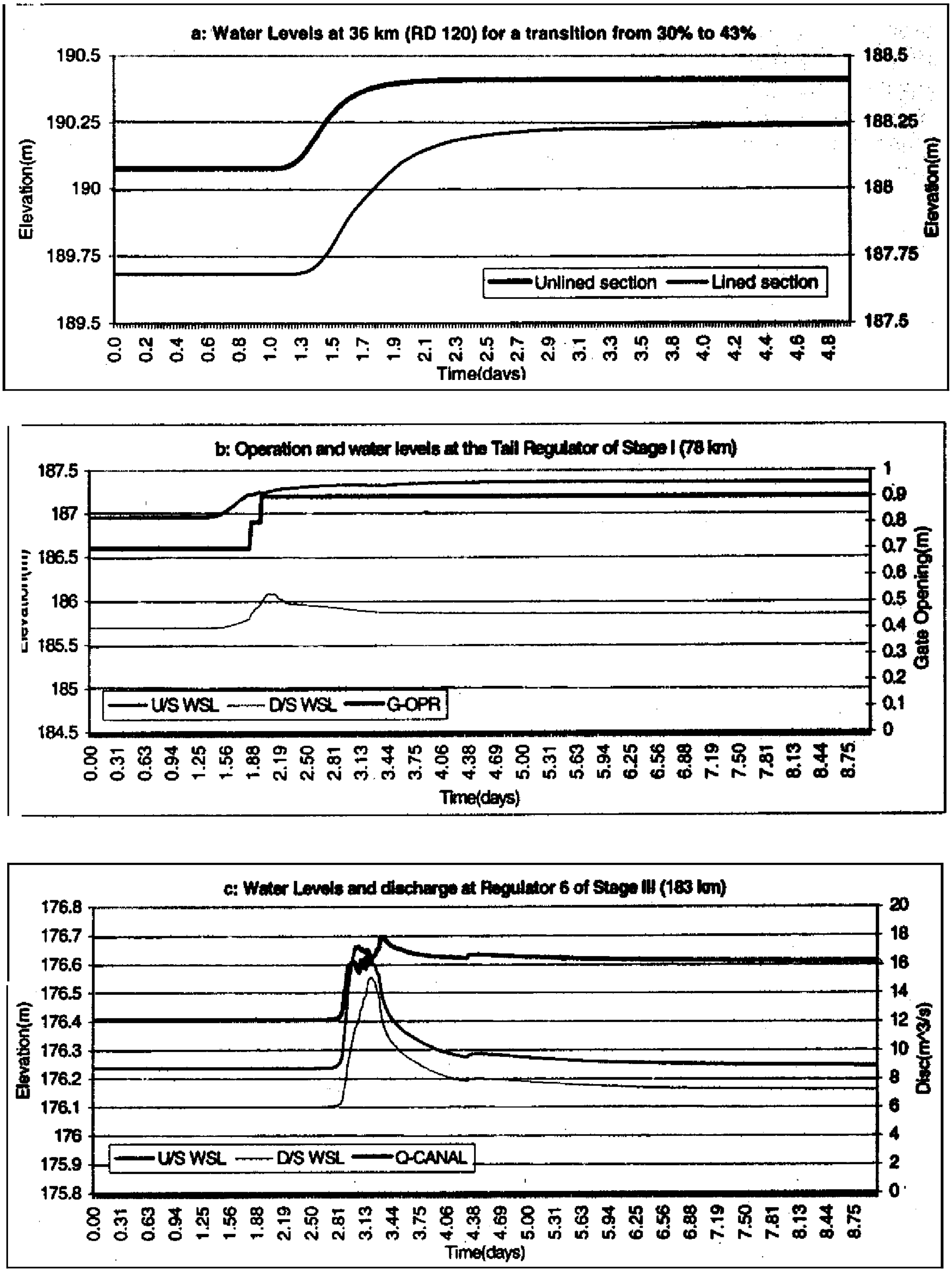
2) The head discharge is increased in 12 hours; its influence starts reaching the tail in three days, which remains quite unstable for more than two days. The canal achieves the next steady state in more than seven days. The **new** water delivery schedule for the secondary system is established during this period.

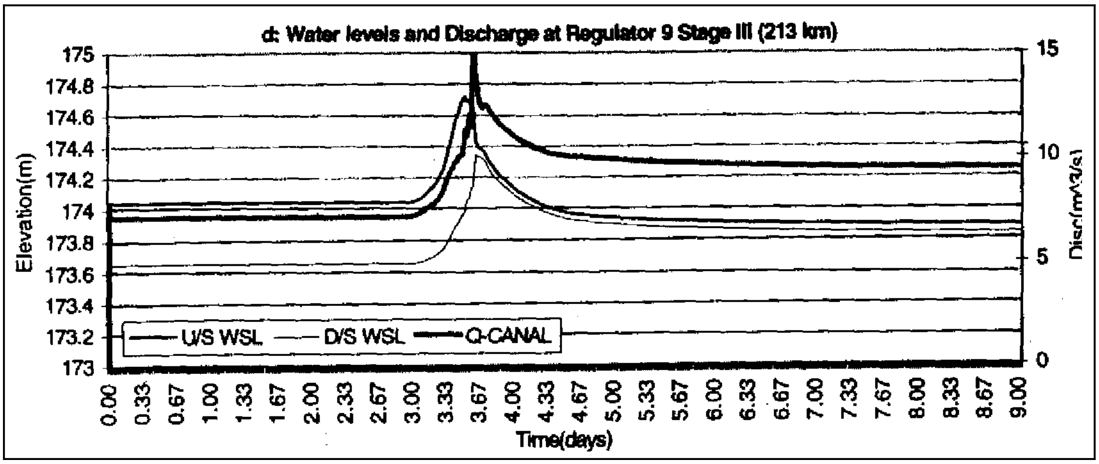
3) The final method of operation should **be** selected according to the priorities of water **supply** objectives and the operational constraints of the system. The required level of stability sometimes leads to the compromises in achieving the required targets, especially during transition. The two methods (Case I and II) of operations are

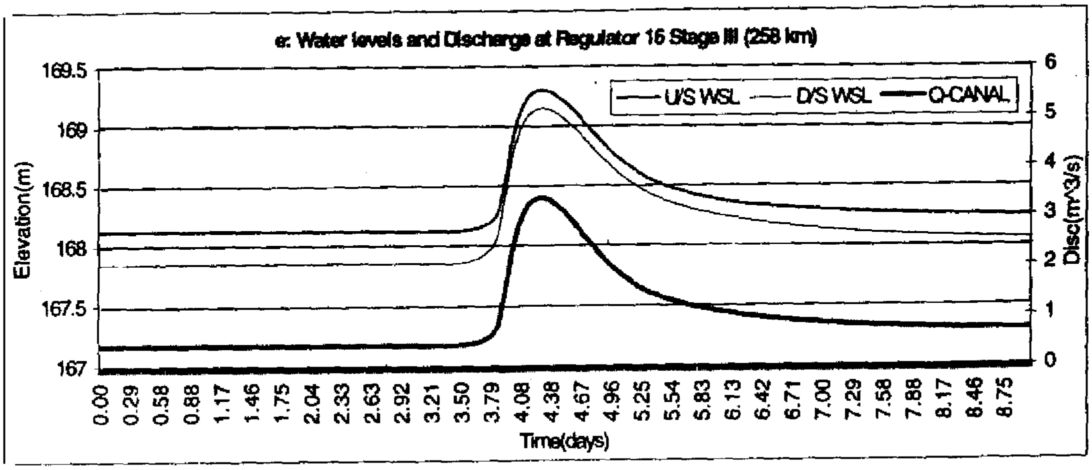
compared here to test the conditions of stability versus equity in the CRBC main canal and distributaries.

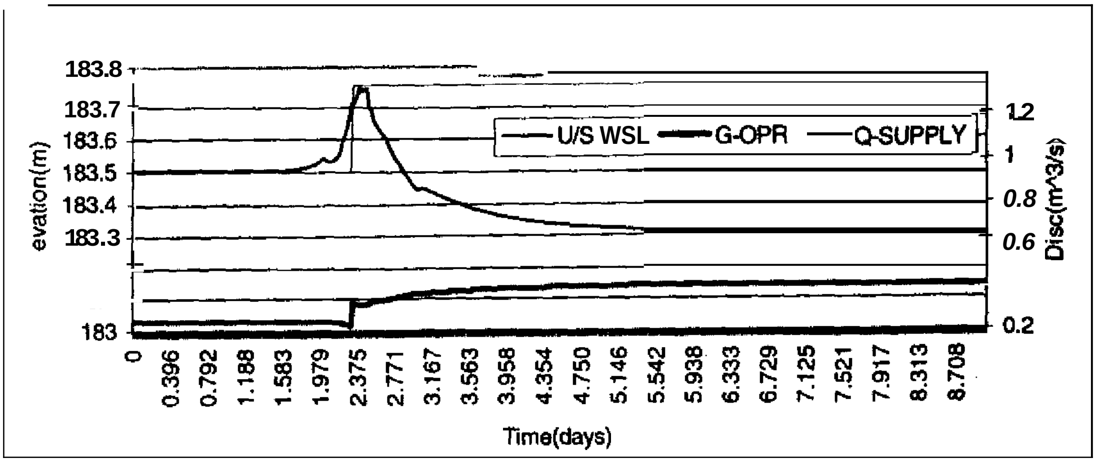
Case I, Increasing the distributary supply while following a schedule rigidly: The discharge of each distributary is increased to the target level, when almost a sufficient working head has been available in a reach. As the canal is not stable, corresponding gate opening could not be set in one step. Hence, it is assumed that the gate operations are adjusted all the time to keep a constant discharge. Figures 8.14 (a-i) show water levels at the cross-regulators and gate opening and discharge at the head of distributaries for these operations.

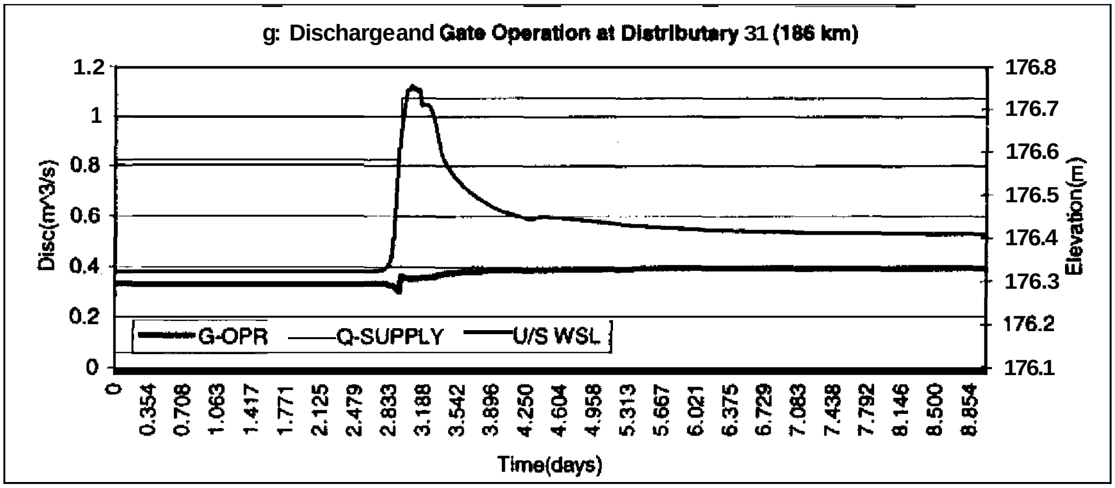
Figure 8.14.1(a-I): Behavior of the canal during a transition from 30 to 43%; the delivery is kept constant.

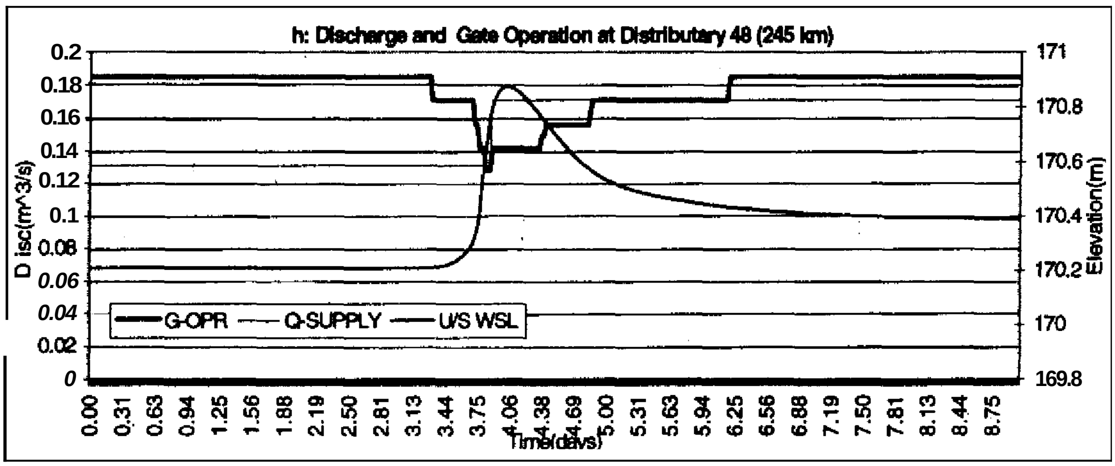


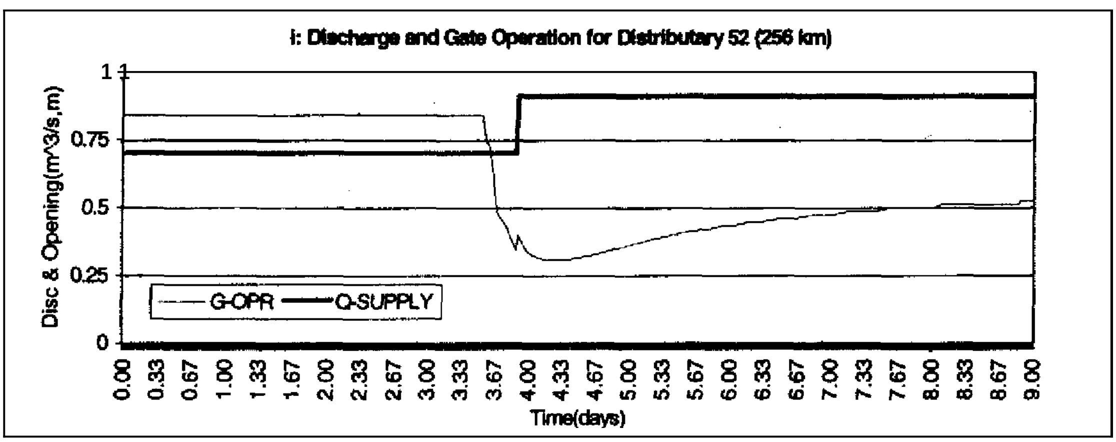

d: Water levels and Discharge at Regulator 9 Stage III (213 km)
Elevation(m)
Time(days)
U/S WSL
D/S WSL
Q-CANAL


e: Water levels and Discharge at Regulator 16 Stage III (258 km)
Elevation(m)
Disc(m^3/s)
Time(days)
U/S WSL
D/S WSL
Q-CANAL


evation(m)
Disc(m^3/s)
Time(days)
U/S WSL
G-OPR
Q-SUPPLY


g: Discharge and Gate Operation at Distributary 31 (186 km)
Disc(m^3/s)
Elevation(m)
G-OPR
Q-SUPPLY
U/S WSL
Time(days)


h: Discharge and Gate Operation at Distributary 48 (245 km)
D isc(m^3/s)
Elevation(m)
G-OPR
Q-SUPPLY
U/S WSL
Time(days)


i: Discharge and Gate Operation for Distributary 52 (256 km)
Disc & Opening(m^3/s,m)
G-OPR
Q-SUPPLY
Time(days)

Case II, Opening the Distributary head regulator before the required working head is achieved: The gates are operated before the final water levels are achieved to share the peak disturbance by the main canal and the distributaries. The supply to the distributaries increases slowly and takes some time to reach the target value. Figures 8.14.2 (a-h) show the water levels and discharge behavior at the control points and for some of the distributaries. The water levels in the main canal are much stable in this case as the fluctuations are shared gradually by all off-takes from a reach.

From the comparison of both cases, it can be concluded that:

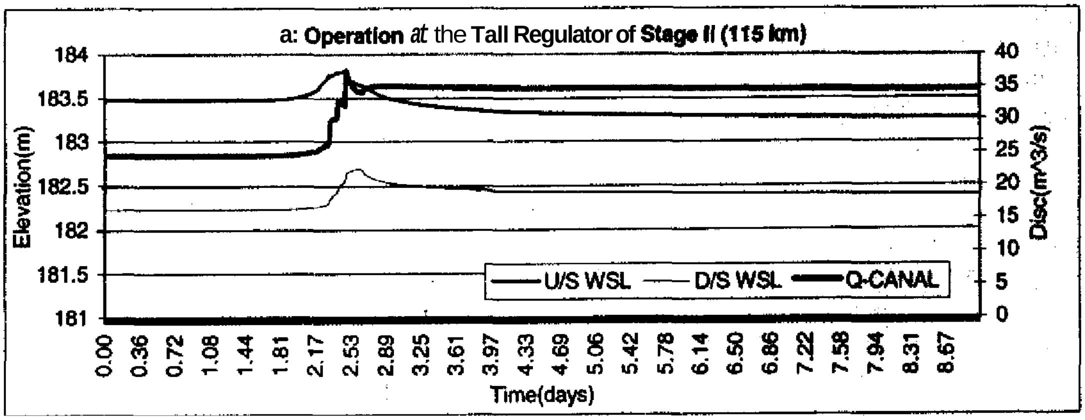
a) The water levels at cross-regulators are more disturbed in Case 1, but stabilize relatively quickly because the outflow is fixed and is affected by the inflow only. The stabilization process is slow in Case II, but the water levels are more stable and the flow moves steadily in each reach.

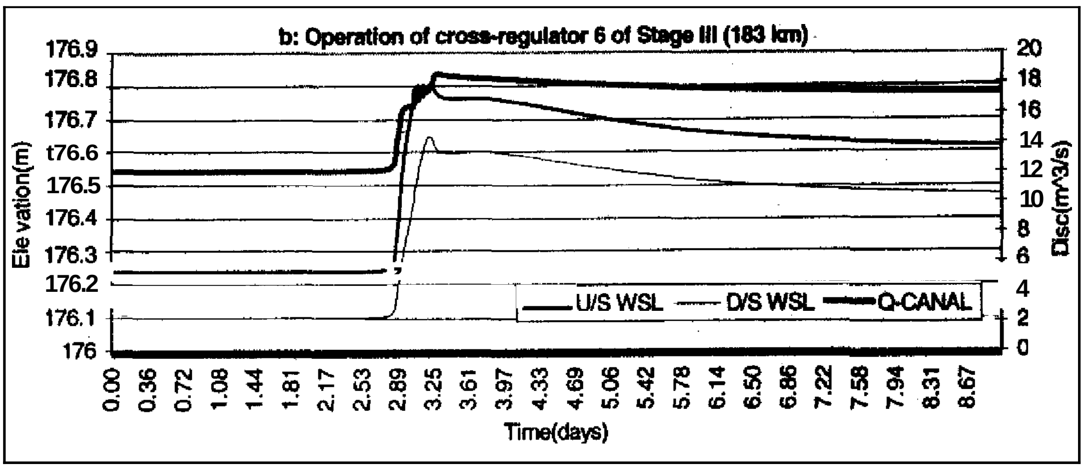
b) The balance of discharge travels towards *the* tail in Case I, while it is shared by all off-takes in Case II. The tail in Case I never gets higher than the target discharge, while in ***Case* II,** it gets more than the full supply for **a** day.

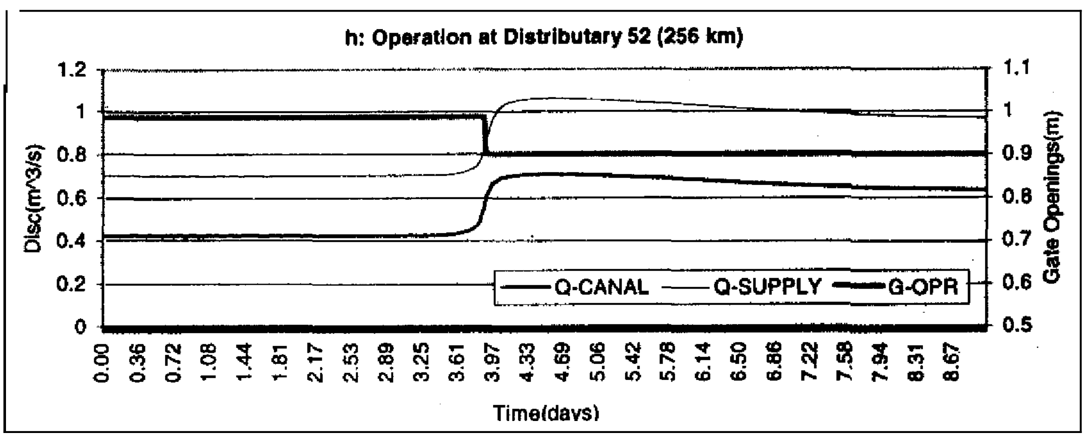
c) The operations required in Case I are numerous, hence, a greater care and involvement is required from the operators. Also, to avoid a bigger disturbance to reach the tail, the timing for all operations need to be carefully computed.

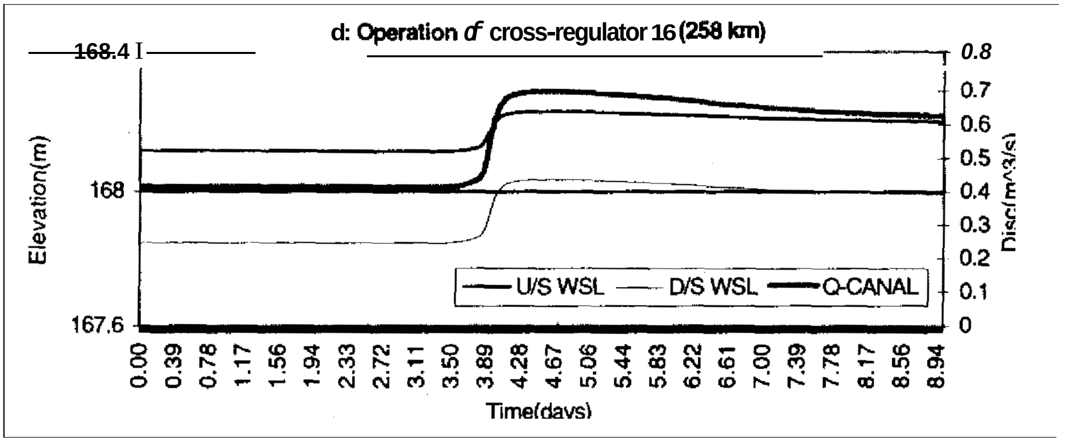
d) The exact quantity of water delivered to each distributary is given in Case I, while in Case II, the distributaries are taking a variable share of water during the transition. Obviously, operations in Case II lead to a less equitable supply during the transition, but better stability for the main canal and distributary structures.

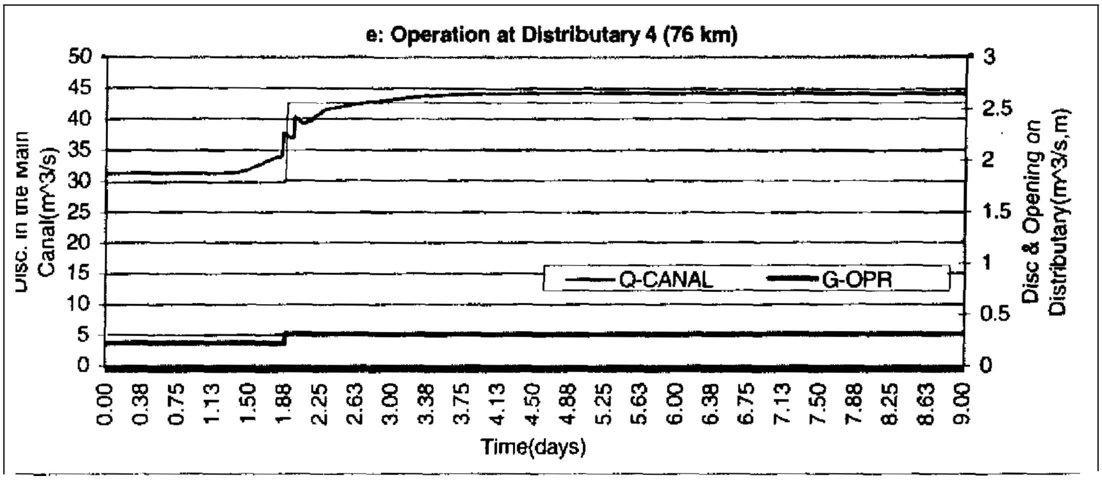
Figure 8.14.2(a-h): Behavior of the canal during a transition from ***30%*** **to 43%; the gate opening of distributary head regulator are kept constant.**

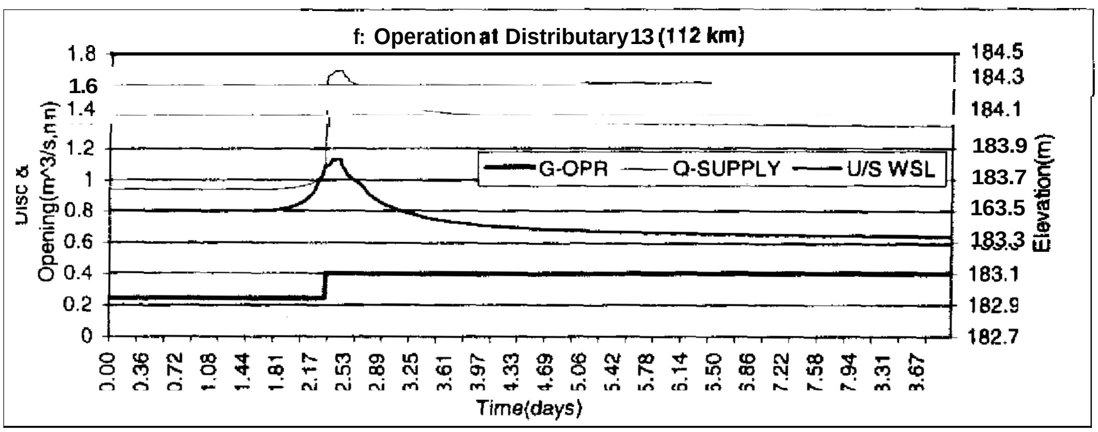


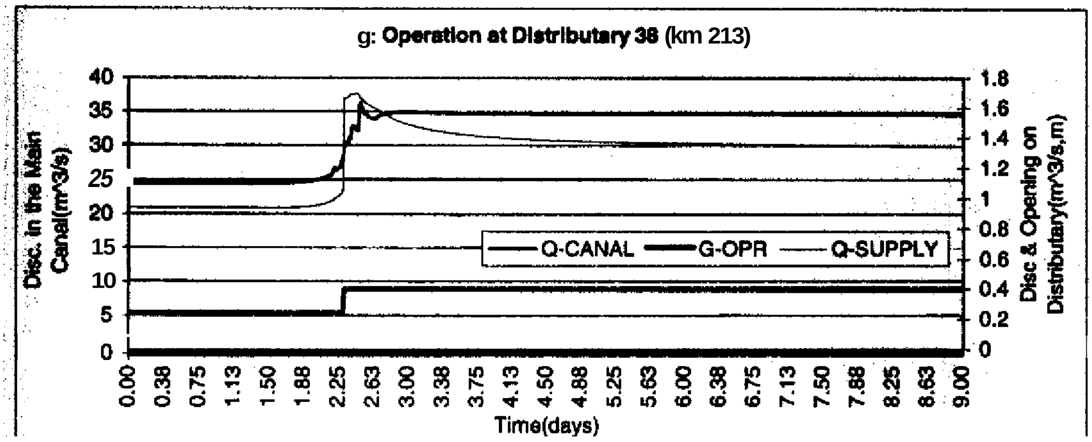


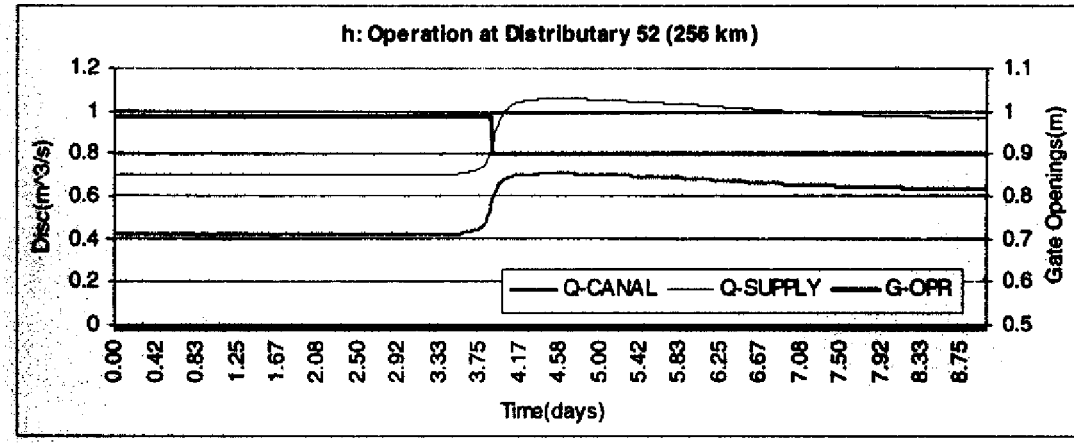



d: Operation of cross-regulator 16 (258 km)
Elevation(m)
Disc(m^3/s)
U/S WSL
D/S WSL
Q-CANAL
Time(days)


e: Operation at Distributary 4 (76 km)
Disc. in the Main Canal(m^3/s)
Disc & Opening on Distributary(m^3/s,m)
Q-CANAL
G-OPR
Time(days)


f: Operation at Distributary 13 (112 km)
Disc & Opening(m^3/s,m)
Elevation(m)
G-OPR
Q-SUPPLY
U/S WSL
Time(days)


g: Operation at Distributary 38 (km 213)
Disc. in the Main Canal(m^3/s)
Disc & Opening on Distributary(m^3/s,m)
Q-CANAL
G-OPR
Q-SUPPLY
Time(days)


h: Operation at Distributary 52 (256 km)
Disc(m^3/s)
Gate Openings(m)
Q-CANAL
Q-SUPPLY
G-OPR
Time(days)

8.7.2 Transition from 43% to *70%*

The behavior of the canal structure and its sensitivity to different operations is the same as shown in the previous case.

Two important characteristics of the transition are:

- A substantial amount of storage is released from Stage I and II cross-regulators, but still, some storage is required in all reaches of these two stages.
- A shift from *a storage* to a *no storage* situation occurs in Stage III.

Hence, the amount of water available in the main canal for distribution is more than the water supplied from the head, which needs to be planned and managed properly. As regulator operation *is* not required in Stage III, stabilization is achieved quickly. To prevent extra water from travelling downstream, all distributaries are adjusted by opening their head-regulators before the final levels are achieved (when about 70% of the water-front is reached to the location of a structure). The water levels and discharges at a few locations are shown in Figures 8.15(a-g).

Figures 8.15 (a & b) show the decrease in the dead storage at Stage I & II cross-regulators, while the extra water is released do'wnstream. The pattern of the pond release depends upon the pattern of the head release from the CRBC with a time lag. The additional water in the upper part of the canal could be utilized for distribution downstream.

The regulation challenge at this point is to release and utilize the discharge downstream at a proper time to avoid unnecessary heading-up in any reach. The escapes provide a very useful utility to achieve a balance quickly. The simulation results indicate that by operating an escape for a couple of hours, the water depth in the upstream and downstream reaches can be maintained reasonably quickly.

8.7.3 Transition from 67% to 100% Inflow

The transition will move the cross-regulators to a non-operating position. Storage management is not required for the final state and all cross-regulators will be full-open, like weir structures. However, during the transition, a major shift takes place in the canal behavior. The transition at some of the structures is shown in Figure 8.16(a-f). The main characteristics of the transition are:

The head inflow needs to be operated slowly, hence the time required to reach the final steady state is relatively long, i.e. more than three days (this is not the time lag, and may be called the response time).

In the previous two transitions, water levels upstream of the regulators were either not raised or slightly increased for all off-takes of the canal. In the present transition, water levels at Stages I & II are still not raised much because they were already at 100% FSL, while the water levels in Stage III are increased from 67% to 100% levels.

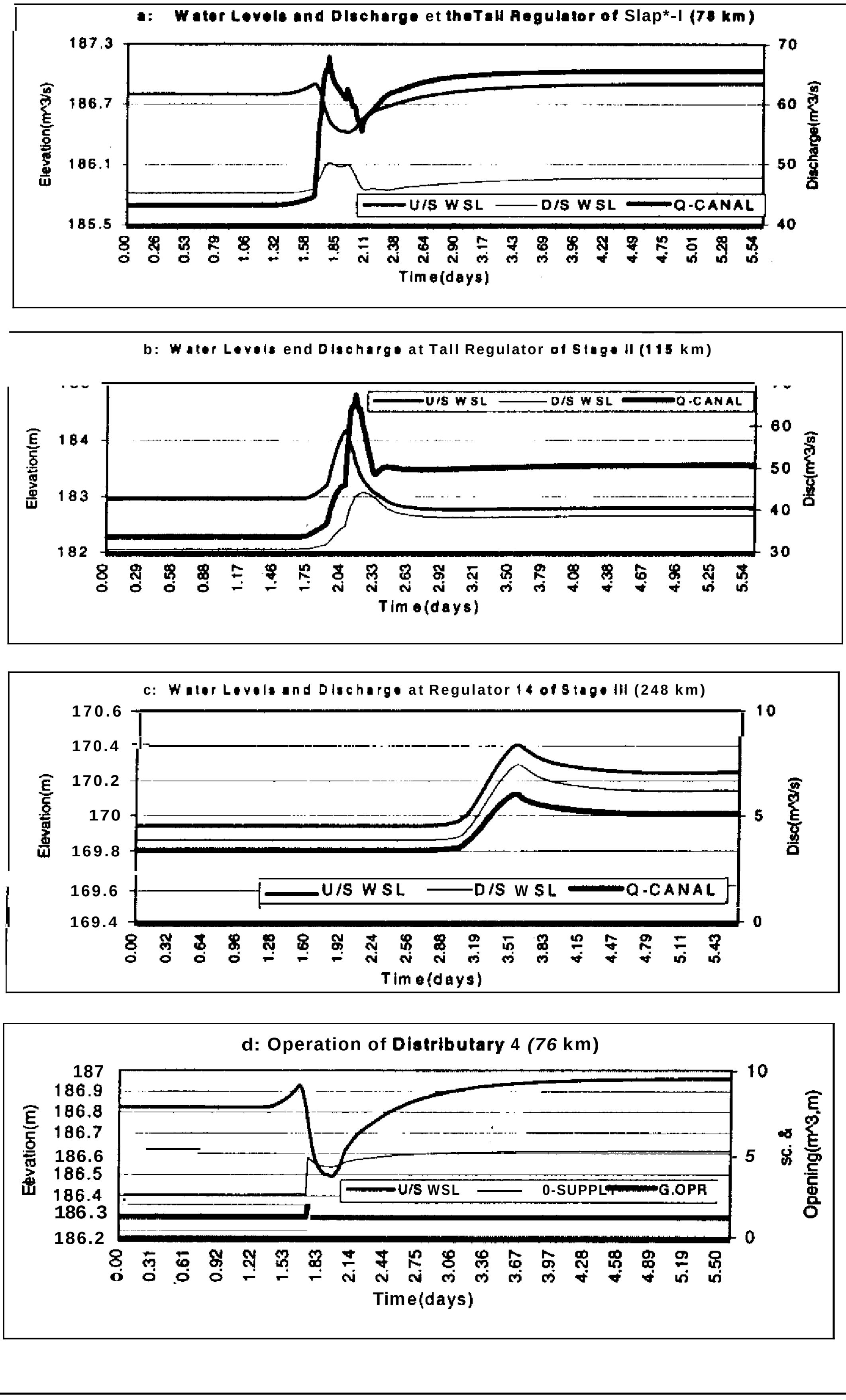
A substantial amount of storage is "released" from the ponding reaches, and the gate operation in Stage I & II are adjusted in many steps.

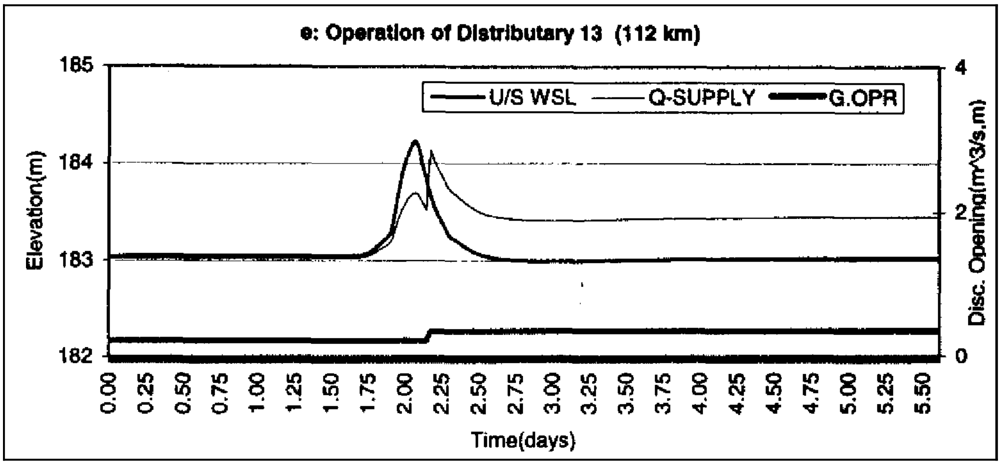
The rate of change of the depth in Stage III is relatively high because the water profile is developed proportionately, The working head for each distributary will increase and new adjustments for the gate operations will be required.

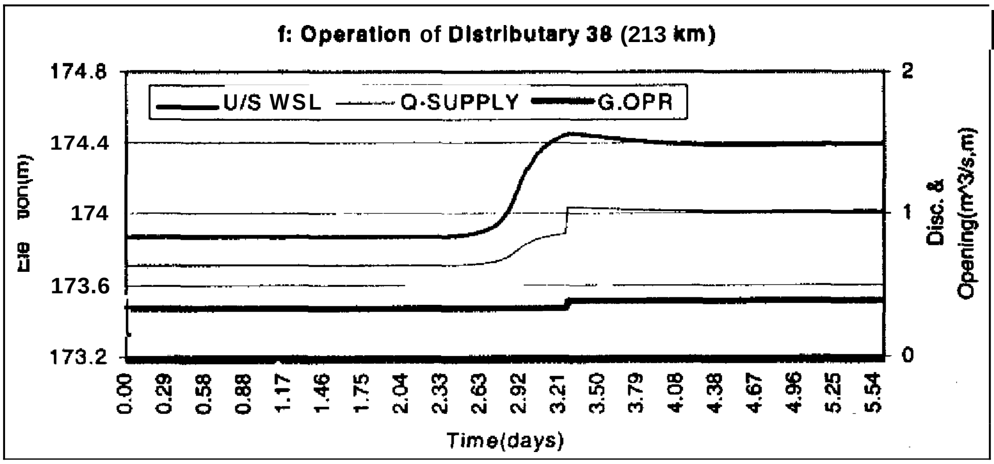
The stabilization time for Stage III will be longer due to the required incremental gate operations.

The tail sensitivity is high during the transitions. The tail would get the net balance of the whole CRBC and during the change of distribution; a combined influence of many unguided operations could reach the tail. The 70% to 100% transition establishes a full supply delivery at all distributaries and direct outlets, balances the upper reach flows towards the tail and limiting the flexibility in the tail reaches.

Figure 8.15: Behavior of the structure during transition from 43% to 70%.




e: Operation of Distributary 13 (112 km)
U/S WSL
Q-SUPPLY
G.OPR
Elevation(m)
Disc. Opening(m^3/s,m)
Time(days)


f: Operation of Distributary 38 (213 km)
U/S WSL
Q-SUPPLY
G.OPR
Ele vation(m)
Disc. & Opening(m^3/s,m)
Time(days)

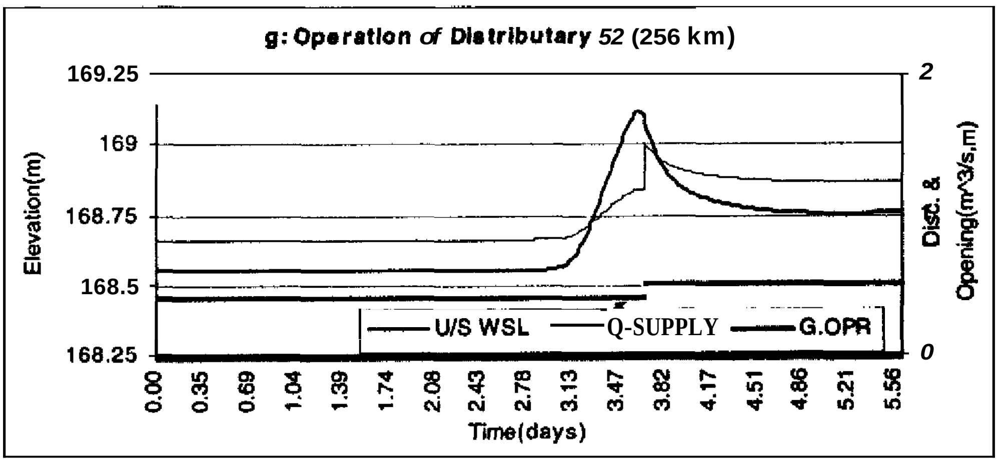
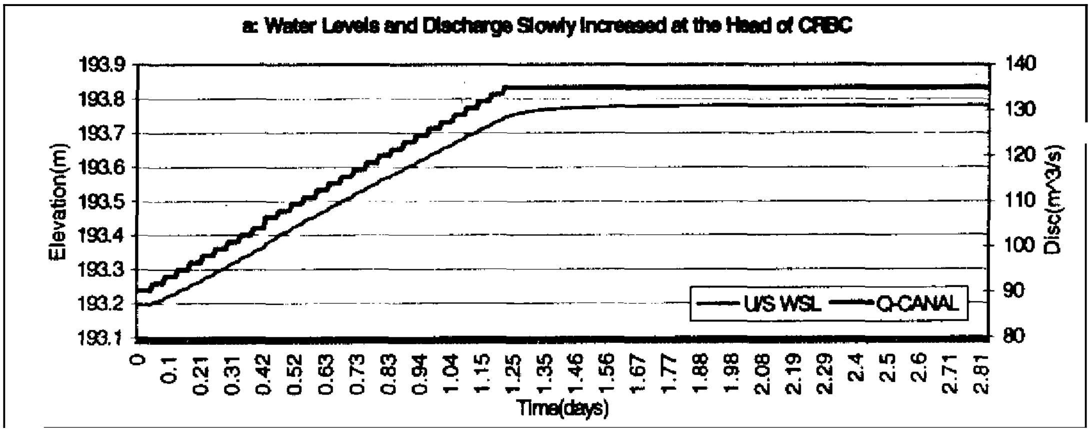
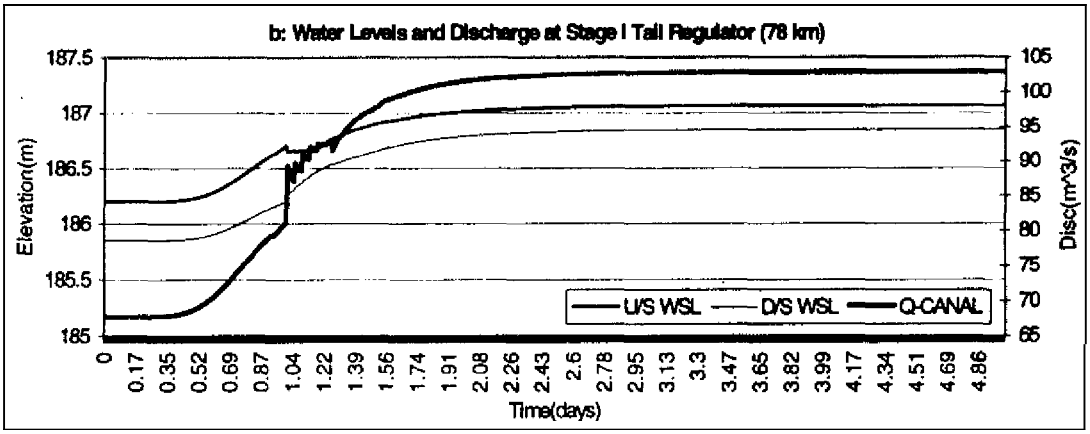
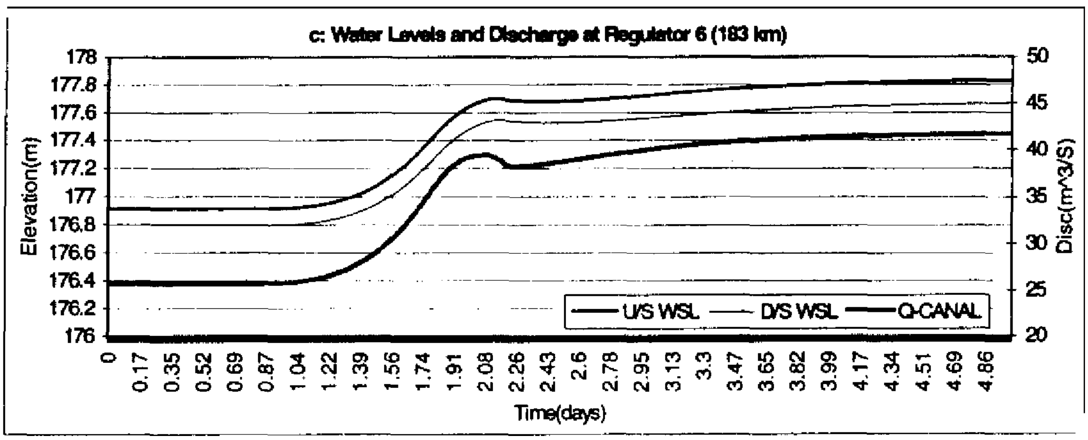
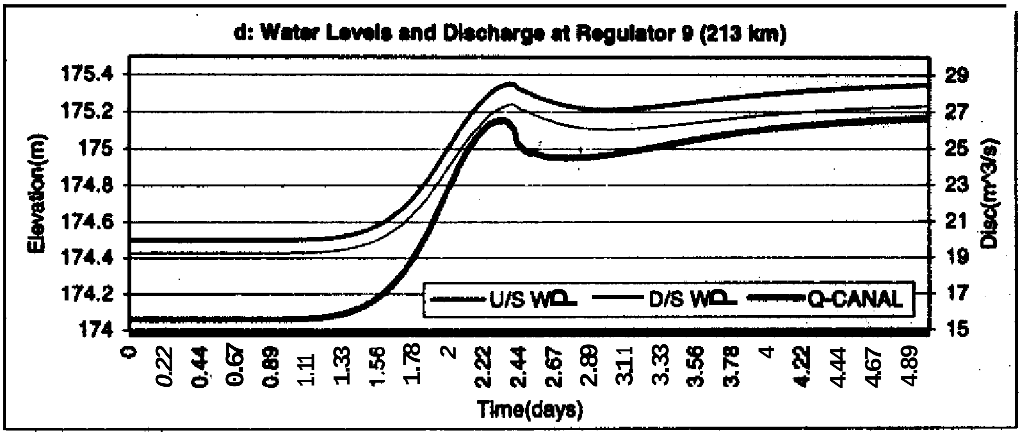

g: Operation of Distributary 52 (256 km)
U/S WSL
Q-SUPPLY
G.OPR
Elevation(m)
Disc. & Opening(m^3/s,m)
Time(days)

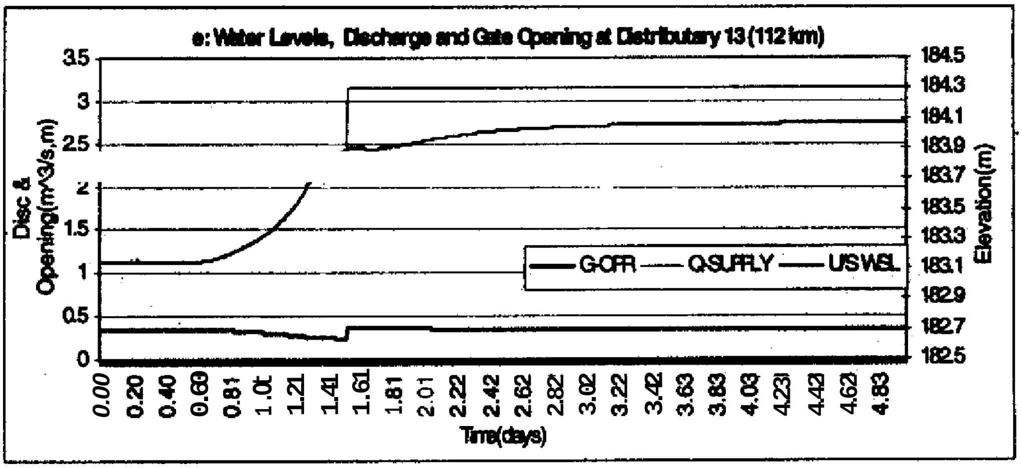
Figure 8.16: Discharge is increased from 90.45 to 135 cumecs (full supply).

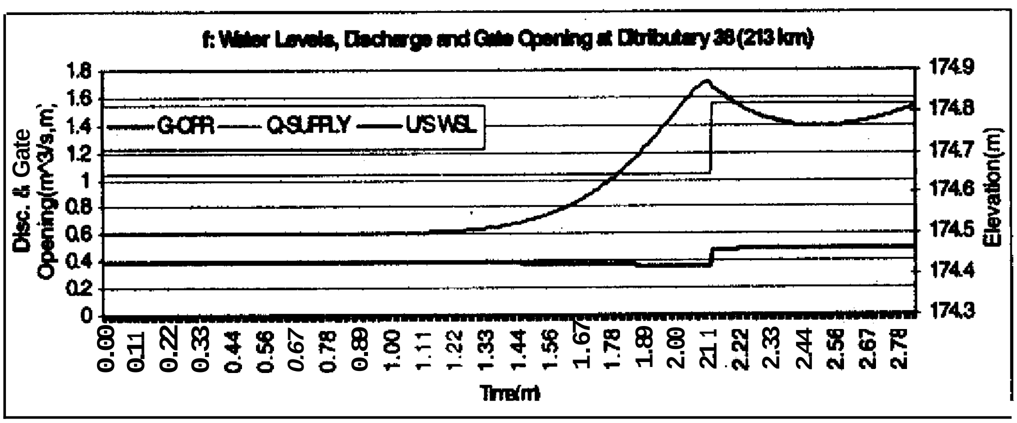







d: Water Levels and Discharge at Regulator 9 (213 km)
Elevation(m)
Disc(m^3/s)
Time(days)
U/S WL
D/S WL
Q-CANAL


e: Water Levels, Discharge and Gate Opening at Distributary 13 (112 km)
Disc & Opening(m^3/s,m)
Elevation(m)
Time(days)
G-OPR
Q-SUPPLY
U/S WSL


f: Water Levels, Discharge and Gate Opening at Distributary 38 (213 km)
Disc. & Gate Opening(m^3/s,m)
Elevation(m)
Time(m)
G-OPR
Q-SUPPLY
U/S WSL

Chapter

9

9. SUMMARY, CONCLUSIONS AND RECOMMEI)AT

9.1 DESIGN AND TECHNICAL

- The Chashma Right Bank Canal off-takes from the Chashma Barrage on the Indus River and travels almost parallel to the river along the contour for 260 kilometers. The first 36 kilometers of the canal are unlined with a slope of 1 in 8,000. A flat slope of 1 in 14,000 has been provided in the next 200 kilometers. The 17 kilometers in the tail reach have a steep slope of 1 in 8,000. Other than depressions, the full supply level in the lined canal is kept at the Natural Surface Level or below at some locations in Stage III. Many cross-drainage structures have been provided as super-passages or siphons for the flood run-off.

- The water allocation schedule of the CRBC is accomplished according to the crop-water requirements of the planned cropping patterns for 60% cropping intensities in *Kharif* and 90% in *Rabi*. The computations are conducted using the US Weather Bureau method for each 10 days. The highest requirement of 8.56 cfs per 1,000 acres of CCA or 8.62 mm/day (4,879 cfs or 138 cumecs head discharge) occurs for one 10-day in October. While the lowest requirement of 2.57 cfs per 1,000 acres of CCA or 1.73 mm/day (1,465 cfs or 41.5 cumecs head discharge) occurs for two 10-days in December.

- To operate the main canal at variable supplies (crop-based), twenty-two cross-regulators have been provided in the main canal, six in the first 108 kilometers of the canal to feed thirty (30) distributaries and fourteen in the remaining 116 kilometers to feed thirty-four (34) distributaries. Six big escape channels have also been provided to manage the extra flow or emergency situations.

- The distributaries off-take only from the left bank of the main canal and traverse towards the river. Due to the steep slope, many falls have been provided along the distributary canals. Distributaries in Stages I and II are unlined, and partially lined in Stage III, and the minimum regulation along these has been provided.

- A roughness coefficient of 0.016 was used for the main canal design in Stages I and II, while 0.018 was used for Stage III. IIMI and the International Sediment Research Institute, Pakistan (ISRIP) have reported a Manning's coefficient of 0.020 in 1991 for the tail reach of Stage I, where sediment deposition was substantial. A sensitivity test indicates a maximum difference of 30 cm between water levels calculated with these two values for the roughness coefficient.

- A preliminary analysis of sedimentation conditions confirms existing trends in sedimentation under the present operational conditions. The analysis indicates that the deposition of sediment is very sensitive to the canal operations The appearance is that canal reaches of Stages I and II, in which heavy water ponding is practiced, act as the sediment traps. From the monitoring of ISRIP, it follows that the sediment load entering the system from the Chashma Barrage is higher than originally assumed levels. The methods should be devised to transport this load through the system in order to prevent its settlement in the canal system. To avoid localized heavy sediment depositions, sudden drops in the sediment transport capacity of the main canal must be avoided.

- To determine the impact of sediment depositions on the canal discharge capacity, two scenarios were evaluated: (a) assuming a uniform deposition along the canal, and (b) the actual situation of 1998: It was found that as a result of the current sediment deposition the canal capacity is reduced by 40%. If the same amount of sediment is deposited uniformly in the affected reaches, the reduction of the discharge capacity was found to be considerably less and only the free board was encroached upon. However, for variable flow conditions, depositions will be maximum where velocities are the minimum.

- The current berm formation in the unlined section is limited to the operational level. The functional lower part of the prism is much narrower than the upper part and a further contraction trend is obvious. Also, in some of the reaches the bed is developing non-uniformly.

- If canal cross-sections are not restored to their original shapes, free board problems may be encountered in the specific reaches of Stages I & I, at the full supply discharge. The critical reaches are: {a) the transition from the unlined to lined section, (b) the tail reach of Stage I, (c) the siphon of Stage I, and (d) the three cross-regulators of Stage II.

- At the design discharge in the CRBC, about 8 m^3/s (280 cusecs) are available in the main canal to compensate for the seepage and conveyance losses. For lower discharges, this amount decreases proportionately.

- To feed the distributaries at low flows, two different approaches have been adopted in the CRBC. In Stages I and II, the working head is increased by heading up the main canal water levels, while in Stage III, the cross-sectional area of the distributary head regulators is increased.

- In Stages II and III, pipe structures are used for distributary aff-takes, whits in Stage I a combination of pipes and square barrels is used. The pipe sizes were kept bigger and the adjustment of discharges is done by gate operation.

- The lift irrigation is provided in the head reaches of Stage III distributaries through sump-wells to command the high level areas.

- An important constraint to water delivery is the high downstream water level at distributary head regulators required to feed high command areas. The resulting submerged flow conditions reduce the flexibility, especially at low flows. In Stages I & II, none of the distributary has a sufficient working head at 67% of the full supply discharge if the cross-regulators are not operated. On the other hand, most of the distributaries can draw 40% to 100% more than their maximum authorization at the full supply discharge in the Main Canal. In Stage III (except for the command of the first two regulators), all distributaries can draw their maximum authorized discharge at 67% of the full design discharge in the Main Canal without cross-regulator operation. For full supply discharge in the Main Canal, they can draw 100% to 200% more than their authorized discharge (Table 6).

- To maximize the commanded area, distributaries are scattered along the main canal, A unique set of water levels is required to feed each distributary for the given flow hydrograph, which makes the system very management-intensive.

9.2 STEADY STATE OPERATION OF THE CANAL

- More than 90% of the structures of the CRBC operate under submerged conditions for the whole of the discharge range.

- According to the Water Apportionment Accord (WAA), the minimum allocation for the CRBC is 41.5 curnecs (1,465 cfs) in December. The WAA does not consider the canal closure period, which is practically observed. If the allocation not used during the canal closure period could be used in other periods, it would be possible to increase the minimum discharge to 62.4 cumecs (2,200 cfs), which, is 43% of the design discharge. WAPDA and the design consultants of Stage III have applied for a modification of the allocations through which the minimum discharge would increase to 43% of the full design discharge, Operational constraints have been determined for both, a minimum discharge of 30% and 43% of the design discharge.

- If cross-regulators are not operated, water levels in the Main Canal for 30% of the design discharge remain below'the crest level of many distributary head regulators (Figure 6.10) and a significant heading-up of water levels (one to three meters) is required to provide a sufficient working head. For 20% of the distributaries, the full supply cannot be obtained (requirement for the rotation) even when the water level upstream of the cross-regulators is raised higher than the design level.

- Proportionate distribution is achieved at 30% of the design discharge if distributary head regulators operate at,free flow, and downstream water levels are lowered to 30% of the full supply levels as, well. However, the supply to tertiary off-takes is adversely effected, some of them being totally out off command.

- Simulation for 43% of the design discharge *(62* cumecs, 2,200 *cfs)* indicates that most of the distributaries can be supplied with the design discharge if the heading-up of the water to the maximum levels is allowed and if a careful rotational plan is adopted. However, eight distributaries (D1, D5, D7, D7b, D8, D8a, D10 and D11a) of Stages I and II can not be supplied with the full design discharge if recommended free board is maintained.

- Proportionate distribution for 43% of the design discharge requires much less ponding, and it is possible to supply all distributaries with 43% of their design discharges. However, the secondary systems (distributaries) are not designed to operate at 43% and their behavior is not analyzed in this study. The current actual operation indicates a need for a more flexible operation of the secondary and the tertiary system.

- At low discharges, the flow velocity decreases substantially in the main canal due to a reduction .in discharge and the heading-up of the water level upstream **of** the cross-regulators. For the first case, the reduction in velocity **is** comparatively **less,** and uniform along the canal, whereas the reduction of flow velocities due to the heading-up is drastic and non-uniform.

- Details for **a** rotational scenario at 50% of the discharge are given in the report. **All** distributaries are supplied in **two** cycles. In Stages **I** and II it **is** necessary to maintain full supply levels to deliver **a design** discharge to **the** farthest distributaries. The grouping of distributaries in cycles **1** and 2 was done to minimize requirements for the manipulation of cross-regulators and to achieve better velocities. The results indicate that D1, **D5,** 07, D7b, D8b, D10 and D11 would not have a margin for a reduction in the diversion capacity of their head regulators.

- **The** current **operation** of the system is **close** to that of demand-based irrigation. **An** analysis of discharge records for a distributary (**#** D3 of Stage **I**) **shows** that **the** total volume delivered during one year is the same as the allocation according to the **WAA.** However, the daily hydrograph is substantially different. **The** minimum supply **is** less than half of the maximum supply, while the **maximum supply** is slightly higher than the maximum authorization for a small period of time, **but,** towards the lower side most of the time.

- An unofficial set-up of communication has been informally established **between** the farmers and operational staff of **WAPDA** and the ID, which is **mostly verbal** and local at the head of each distributary. **It** includes the process **of** placing indents **and** giving instructions **for** the day-to- day release of discharges.

9.3 THE HYDRAULICS OF UNSTEADY STATE AND TRANSITIONS

9.3.1 Hydraulic Sensitivity of the Structures

The sensitivity of the cross-regulators, escapes and distributary **head** regulators **was** evaluated for different operating conditions.

9.3.1.I X-regulators

- The sensitivity of a cross-regulator is the minimum when **it** functions as a weir. According to the simulated design parameters, cross-regulators in Stages **I** & II function as weirs at discharges higher than **95%** of the design discharge, while in Stage **III** the cross-regulators function as **a** weir at flows higher than 67% of the design discharge.

- **At** lower discharges, the gates of the cross-regulators are submerged orifice and highly sensitive to the downstream water levels. This is **caused by** the fact that in order to reduce the head **loss** to **a** minimum, cross-regulators were not provided with a level-drop. In most *of* the reaches of Stages II & **III**, even the distributaries just downstream of the regulators do not have sufficient working **head** at **43%** of the discharge. Hence, backwater reaches the upstream cross-regulator, resulting frequently in a chain of cross-regulators influencing each other. **At** the same time, distributary head regulators are also submerged. **As** a result, the operation of one of these structures influences the flow through a whole chain of cross-regulators and head regulators.

- The influence of the release or depletion of the storage upstream of a cross-regulator can be quite considerable. It was found that many structures had to be operated as a response to such a release and it took a few days to achieve a new steady state situation. The higher the storage upstream of a cross-regulator, the more sensitive such a cross-regulator is, and the bigger its influence downstream.

9.3.1.2 *Escapes*

The seven escape channels of the CRBC provide an important management tool and can **be** used to cushion and control the responses of unplanned operations and emergency high flows or excessive water situations.

- The operation ranges for the escapes are well-defined; the influence of an escape being more effective in the downstream reaches, as could be expected.
- In the case of a higher than required discharge in a reach, the escape can **be** used to maintain the water depth; however, this option is limited to reaches with escapes.
- In the case of rainfall or other emergencies, escapes could play an important role in **view** of their large capacities.
- The flow velocity generated by the operation of escapes should not be higher than the maximum permissible velocity for the canal. This limits the possibilities for using the **spillways** to flush sediments.

9.3.1.3 *Live reach storage*

The unsteady state analysis for different scenarios indicates that the most critical parameter/situation to be managed in the CRBC is the reach storage (ponding). Especially in the long canal reaches of Stages **i** & **II**, large volumes are stored, which are consumed during **the** filling up and added to the water balance during transitions to the higher supplies. Three situations related to the management of this storage are:

a) The dead storage is at a maximum at low flows. The filling times and volumes **would** need to **be** computed (or assessed practically) at the start of operations. One example is presented in the report.

b) When moving from lower to higher supplies, the wedge storage is to be released. This could be utilized in the system by appropriate operations of the distributaries.

c) The storage could also be released *to* the escapes and diverted **back** *to* the Indus River. The option is favorable to avoid the fluctuations of the levels in the main canal.

9.3.2 Unsteady State Behavior of the Main Canal Conveyance System

The responsiveness of main canal reaches and structure is analyzed for critical situations, like filling up of the canal, flow transitions, storage depletion and the unplanned operations of cross-regulators and escapes.

- The computation procedures used to assess the response and lag times are presented in the report. The computed lag times at each cross-regulator for 30%-43% and 70%-100% are presented. It must be considered that the simulation is carried out under the ideal conditions for the head release and gate operations, which is difficult to be achieved 100% in the field. However, the analysis provides basic information as well as the first assessment of time lags and canal filling.

  i. Under normal flow conditions, time lags are proportional to the average flow velocity and the filling requirements. The number of operations required to be performed on the cross and head regulators would be many due to their high sensitivity.

  ii. The average head-to-tail time lag for the CRBC is about 3 days.

  iii. For a reduction in the canal discharge the response time is shorter than for an increase in discharge due to a higher wave propagation velocity resulting from the initial, larger flow depth.

  iv. A minimum stabilization time can be achieved if the regulators are operated when 40%-50% of the wave (turbulence) is received at its location.

  v. The computed (by the model) lag plus stabilization time (net response time) at a flow transition from 43% to 70% is about a week for the main canal.

  vi. The escapes could be used to achieve a quicker stabilization at high flows if some wastage of water (for the system) can be afforded.

- The water progression and the stabilization times during the filling up of the canal are computed for 30% of the supply operations. The discharge in the canal is increased from 3 cumecs to 41.5 cumecs (125 to 1,465 cusecs) in two days, at a maximum rate of 10 cm per hour. A proportionate delivery for all distributaries could be started after four hours. The filling hydrograph at the end of each day increases in pond levels and the stability achieved in each reach are shown graphically. The final results provide the lag and stability times for all reaches and structures.

9.3.3 Canal Behavior During Transition

The simulation of flow transitions provides two additional pieces of information for the operation and management of the main canal.

- The CRBC has a single and long main conveyance system, which has not been provided with a branched network (physical division of the canal at a specific location through proportionate dividers). Hence, excess, shortage or variability could not be sub-divided or dampened at a location. The balance impact of all actions along the canal could travel downwards without self- regulation. Also, the net influence of all instabilities can make the tail quite vulnerable. Physically, the tail of the CRBC is a single distributary of about four cumecs, which is a small discharge to compensate the expected variability.

- The overall instability and the risk of tail variability increases, if a schedule is strictly followed and the delivery to a distributary is started at a fixed time. In that case, the final state is achieved in less time, but the magnitude of fluctuations is considerable. When distributaries are used as a cushion, the fluctuations are dampened, which means operating distributary gates before the required water depth is achieved. The time to reach the final stability is longer, but fluctuations are less in this case,
- The escapes could also be used to maintain or achieve water levels, but the resulting loss of water must also be accepted during *Rabi* when required flow transitions are relatively frequent.

9.4 RECOMMENDATIONS

All technical and managerial problems and constraints faced by the CRBC may not be solved at this stage. Nevertheless, it is high time for the policy makers to clear the ambiguity about the main canal allocation, bridge the gap between diversified approaches for the design and operations and start shifting from temporary arrangements to permanent procedures.

The following recommendations are based on the hydraulic analysis of the flow conditions in the canal, which were summarized in the previous section. Again, it should be remembered that all diagnostics were based on the design conditions and that the actual canal operation in Stages I and II was used as a reference to confirm some of the results.

1. The 10-daily water allocation to the CRBC is the most important basic parameter for the selection of design and operation criteria. The minimum allocation of 30% may be sufficient *to* meet crop water requirements in *Rabi*, but this discharge was not used for the design of Stage I, Stage II or Stage III. The hydraulic analysis (see summary) indicates that:

   - The CRBC can not be operated at 30% of the full supply discharge.

   - The higher supplies in *Kharif* should not **be** compromised because the sediment load is the maximum during that period and a drop in velocity could be deterrent. Secondly, demands of the farmers are even higher than the maximum authorization during *Kharif* and a reduction in the upper limit may badly influence their production margins.

   From an operational point of view, two alternatives should therefore be considered:

   i. *A minimum allocation of **43%** (60 m^3/s or 2100 cfs):* This alternative requires least adjustments of 10-daily allocations and the Stage III design is based on it. **A** *week-on week-off* rotation would deliver 80% of the full supply discharge into a distributary during one out of two weeks.

   ii. *A minimum* allocation **of** 50% *(69 m^3/s or 2440 cfs)*. For this alternative, the operational flexibility for Stages I & II will increase. It would require relatively more adjustments in 10-daily allocations. **A** *week-on week-off* rotation would deliver 100% of the supply discharge into the distributaries during one out of two weeks. The design of Stage III **would** need to be reconsidered.

   It must be considered that the actual water demand could be less than 50% or 43% and some of the farmers might refuse to receive water. Both alternatives, therefore, need escapes although *Moghas* are ungated and farmers are not allowed to close *Moghas*.

2. The supply to eight distributaries (D1, D5, D7, D7b, D8, D8a, D10 & D11) in Stages I and II is made difficult or is not fully possible at lower discharges due to the high water levels required downstream. For a rotational supply at 43% of the design discharge, these distributary structures could not be supplied without an encroachment on the free board upstream **of** the relevant cross-regulator. At 50% of the full supply discharge, water levels are just at the margin, while the gate openings are at the maximum. Even

at 67% of the design discharge or higher, substantial ponding is required to supply these distributaries. A readjustment to feed the upstream reaches of these distributaries must be considered.

3. The design assumptions adopted for Stages I and II and those adopted for Stage III are too far apart. The present study could not provide design alternatives as the constraints posed by super-passages, the selection of escapes and the location of distributary off-takes are often more important than the hydraulic justification of the structures. However, it is recommended that the distributary head and cross-regulators (locations and dimensions) provided in Stage III should be rechecked with reference to the analysis provided in this report. This may help to avoid a Lower Swat type situation where most of the cross-regulators are not operated and where the ability of the secondary canals to draw more than their share poses management difficulties.

4. The sediment deposition of about 3 meters (10 feet) in a few sections of the canal has reduced the capacity substantially, could be hazardous for the canal lining and suggests that the present operation procedure must be changed without waiting for the completion of the CRBC Stage III. Currently, *Kharif's* operational discharge is less than the minimum water allocation recommended for *Rabi.* This is causing heavy silt deposition and the actual capacity of the canal has been substantially reduced. Therefore, it is recommended to operate the canal during *Kharif* at high flows. The three big escapes, which are available in Stages I & II should be used to divert water back to the Indus River. This would also help the berm formation in the unlined section of Stage 1.

5. The submerged flow through regulators and reach storage (structure to structure) makes water levels very sensitive to the changes in the flow pattern. The time required for stabilization of the flow is longer, with a head-to-tail time lag of more than three days. As a result, it will require several days for flow conditions in the canal to stabilize following a change of discharge at the head of the canal, and unstable conditions would occur for an important part of the ten-day period for which the adjustment was made. To avoid frequent disturbances, it is recommended to combine 10-daily periods as much as possible to provide a constant flow over longer periods (20-30 days).

6. The deposition of sediment is not an issue in many systems in Europe and America, and is, therefore, often not considered in their design criteria. In those systems, a big drop in the velocity is acceptable. For the Indus Basin, such changes in velocity need to be observed and the use of the critical minimum velocity is an acknowledged design parameter for systems, where the regime theory is strictly adopted to allow the functioning of canals within a range of 70% to 110%. The design and operating assumptions for the CRBC (for Stages I & II as well as for Stage III) have not considered the velocity factor at all. An analysis of sedimentation in the CRBC indicates that an irregular velocity profile is a more contributing factor for sediment deposition than low velocities. Hence, heterogeneous velocity profiles *in* canal reaches must be minimized as much as possible.

7. Constant depth operation is a standard method for the operation of upstream control structures. A fixed water level is maintained upstream of the regulators for a season, or

[4] The discussion with local WAPDA staff indicates that they are aware of the problem. The solutions indicated during these discussions are the provision of direct outlets *for* command areas of the upper reaches of these distributaries, or the provision of lift pumps to feed the reaches. In the case of Distributary 1, the pipe size could be increased..

a year, while small fluctuations in the discharge are **allowed.** In the case **of** the **CRBC,** fluctuations **between** the minimum and the maximum discharges are quite large, and the use of a single water level target would cause prolonged and unnecessary water storage, and consequently, a drop in velocity. Therefore, it **is** recommended **to** adopt three or four different water levels at cross-regulators for the discharge ranges of, for example, **c 50%, 50%-67%,** 67%-80% and **80%-100%. The** variable depth operating criteria would require **a** higher management input, which could not **be avoided in case of** CRBC, **as** the system is management-intensive **by** design.

8. The current operation of cross-regulators in Stages **I** and **II** is an important tool to understand the flexibility and constraints of the system. The deliveries to **the** distributaries are not managed according to any given schedule **and** follow the **demand** pattern to a large extent. An interesting *"communication **and control network**"* **is** in place, where, apparently all parties are happy, i.e. **WAPDA,** the ID and farmers. The analysis and understanding of **this** situation would provide valuable information about the operational **as** well **as** the institutional dynamics **of** the system. Unfortunately, the current data recording set-up is totally unreliable, and the information **could** not be readily used for the analysis. It is the time *to* start implementing monitoring **and** evaluation procedures and collecting data of daily operations in **a standard** format.

9. The study has provided quite a comprehensive analysis of the hydraulic behavior, flexibility and limitations of all canal structures. **Many** sensitivity scenarios have been developed and compared, which **provides** the data and comparisons for different ranges of operations. Guidelines are provided for managers for storage computation, filling behavior of the canal, time lags, functioning of structures and off-takes, **and** adjustments during transitions. The major part of the model software and work done during this study is now available with **WAPDA.** It is recommended for **WAPDA** to appoint a professional to **use** this model for further evaluation of the **IIMI** recommendations, and try *to* learn from the experience of the CRBC. This type of exercise **will** improve the confidence and knowledge **of** the real managers **of** the system and **help** them to solve **some** of the conceptual and practical contradictions **faced by** the CRBC system.

10. The CRBC will require a strong communication setup, as well *as* a **network** to manage and transfer information within the system. The proper monitoring and evaluation procedures could not be established without that.

11. The CRBC is a high water allocation system, basically designed for full supply operations. For such a system, it **is** normal to **have** operational constraints at extreme supplies, **while** operating different levels (primary, secondary & tertiary). The minimum supplies required in ***Rabi*** for the operation of the system could **be** higher than the command area requirements. The proportionate distribution at all **levels** is ***a*** *control-*intensive system and **has** not been recommended for the CRBC. Nevertheless, to comply with a shift in allocation, a shift in operation **is an issue** for further action research. The following indications provide **the** justification:

- The rotation **is** not hydraulically favored along the main and secondary canals, **while** these canals **could be** better operated on proportionate supplies,

- The water demand of Stages I and II is higher than the authorization in *Kharif* and about thirty percent of the full **supply** in *Rabi.* This means that the *Rabi-Kharif* demand ratio is the same as the allocation ratio and an effort to minimize this gap will not be in the favor of the command area.

- The secondary system is currently operating at the request of farmers, and at much **lower** flows in *Rabi*. **Even** for the constant head release for the CRBC main canal, the distributaries **are** operated on day-to-day variable flows.

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